

STRATIGRAPHICS

PROJECT NAME:Stratford Army Engine Plant

PROJECT NUMBER:99-120-050

R2DATE: 5-10-1999 TIME:13:53:20.59

SOUNDING NUMBER:CP-9912

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (Nf1)
100.5	192.0	145.2	2.0	4753	Dense, Silty sand to sandy silt	37.40	60-80				53-79	40-60
101.0	194.3	146.8	2.0	4752	Dense, Silty sand to sandy silt	37.40	60-80				53-79	40-60
101.5	189.7	143.1	1.9	4823	Dense, Silty sand to sandy silt	37.40	60-80				53-80	40-60
102.0	196.3	147.9	1.9	4863	Dense, Silty sand to sandy silt	37.40	60-80				53-80	40-60
102.5	176.4	132.7	3.40	4830	Dense, Silty sand to sandy silt	37.40	60-80				53-80	40-60
103.0	178.4	134.8	3.15	4835	Dense, Silty sand to sandy silt	40.42	60-80				53-80	40-60
103.5	173.0	129.8	3.46	5032	Dense, Silty sand to sandy silt	37.40	60-80				53-80	40-60
104.0	147.8	110.8	2.74	5112	Dense, Silty sand to sandy silt	37.40	60-80				53-80	40-60
104.5	202.2	151.3	3.13	5193	Dense, Silty sand to sandy silt	40.42	60-80				40-53	30-40
105.0	178.8	133.6	4.00	5373	Dense, Silty sand to sandy silt	37.40	60-80				53-80	40-60
105.5	161.6	120.6	2.61	5173	Dense, Silty sand to sandy silt	40.42	60-80				54-80	40-60
106.0	199.3	148.5	3.72	5337	Dense, Silty sand to sandy silt	40.42	60-80				40-54	30-40
106.5	169.4	126.1	2.99	5316	Dense, Silty sand to sandy silt	40.42	60-80				54-80	40-60
107.0	197.1	146.5	1.6	5273	Dense, Sand to silty sand	40.42	60-80				54-81	40-60
107.5	197.8	146.9	2.98	5351	Dense, Sand to silty sand	40.42	60-80				54-81	40-60
108.0	203.1	150.6	3.06	5537	Dense, Sand to silty sand	40.42	60-80				54-81	40-60
108.5	230.9	171.1	2.21	5524	Dense, Sand to silty sand	42.46	60-80				54-81	40-60
109.0	252.4	186.7	3.03	5434	Dense, Sand to silty sand	42.46	60-80				54-81	40-60
109.5	191.3	141.3	3.83	5507	Dense, Silty sand to sandy silt	40.42	60-80				54-81	40-60
110.0	176.7	130.4	3.85	5706	Dense, Silty sand to sandy silt	37.40	60-80				54-81	40-60
110.5	194.3	143.2	3.73	5748	Dense, Silty sand to sandy silt	37.40	60-80				54-81	40-60
111.0	192.7	141.8	3.75	5929	Dense, Silty sand to sandy silt	37.40	60-80				54-81	40-60
111.5	196.2	144.2	3.50	5813	Dense, Silty sand to sandy silt	40.42	60-80				54-82	40-60
112.0	216.1	160.1	2.95	5770	Dense, Sand to silty sand	40.42	60-80				54-82	40-60
112.5	226.3	165.9	3.52	5770	Dense, Sand to silty sand	40.42	60-80				55-82	40-60
113.0	218.2	159.8	3.44	5899	Dense, Sand to silty sand	40.42	60-80				55-82	40-60
113.5	196.0	143.3	3.83	5713	Dense, Silty sand to sandy silt	40.42	60-80				55-82	40-60
114.0	172.7	126.1	3.69	5714	Dense, Silty sand to sandy silt	37.40	60-80				55-82	40-60
114.5	190.5	138.9	3.50	5956	Dense, Silty sand to sandy silt	40.42	60-80				55-82	40-60
115.0	195.5	142.5	3.41	5953	Dense, Silty sand to sandy silt	40.42	60-80				55-82	40-60
115.5	188.5	137.2	3.73	5844	Dense, Silty sand to sandy silt	37.40	60-80				55-82	40-60
116.0	168.8	123.4	4.39	5740	V dense, Silty sand to sandy silt	37.40	80-100				55-82	40-60
116.5	138.0	100.2	4.47	5771	Hard, Silty sand to sandy clay	37.40	80-100				55-83	40-60
117.0	131.2	95.1	4.48	6074	Hard, Gr cl sand to gr sa silt			30	8.73	8.94	55-83	40-60
117.5	125.9	91.1	4.26	6051	Hard, Gr cl sand to gr sa silt			30	8.28	8.96	55-83	40-60
118.0	124.5	90.1	4.17	6114	Hard, Gr cl sand to gr sa silt			30	7.92	8.53	55-83	40-60
118.5	124.0	89.6	4.42	6320	Hard, Gr cl sand to gr sa silt			30	7.83	8.33	55-83	40-60
119.0	121.1	87.4	4.24	6144	Hard, Silty sand to sandy clay			30	7.80	8.84	55-83	40-60
119.5	156.8	113.0	3.83	6271	Dense, Silty sand to sandy silt	37.40	60-80				55-83	40-60
120.0	189.5	136.4	3.71	6276	Dense, Sand to silty sand	40.42	60-80				56-83	40-60
120.5	297.1	213.6	3.72	6205	Dense, Sand to silty sand	42.46	60-80				56-83	40-60
121.0	294.5	211.4	3.99	6193	Dense, Sand to silty sand	42.46	60-80				56-84	40-60
121.5	564.1	404.6	0.13	4773	Dense, Sa gravel to gr sand	+46	60-80				84-138	60-99

Notes:

* Indicates lightly overconsolidated soil

** Indicates heavily overconsolidated or cemented soil

Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.

Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.

Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

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SOUNDING NUMBER: CP-9912

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1')
1.0	Prepunched To 8.50'												
1.5	Prepunched To 8.50'												
2.0	Prepunched To 8.50'												
2.5	Prepunched To 8.50'												
3.0	Prepunched To 8.50'												
3.5	Prepunched To 8.50'												
4.0	Prepunched To 8.50'												
4.5	Prepunched To 8.50'												
5.0	Prepunched To 8.50'												
5.5	Prepunched To 8.50'												
6.0	Prepunched To 8.50'												
6.5	Prepunched To 8.50'												
7.0	Prepunched To 8.50'												
7.5	Prepunched To 8.50'												
8.0	Prepunched To 8.50'												
8.5	Prepunched To 8.50'												
9.0	23.9	28.4	0.31	1.2	524	Loose, Silty sand to sandy silt	27-31	20-40				03-05	04-06
9.5	15.0	17.7	0.29	1.5	491	Loose, Silty sand to sandy silt	27-31	20-40				02-03	02-04
10.0	17.9	21.0	0.26	1.3	557	Loose, Silty sand to sandy silt	27-31	20-40				03-05	04-06
10.5	15.7	18.4	0.33	1.7	550	Loose, Silty sand to sandy silt			15	2.01	0.66	03-05	04-06
11.0	21.2	24.7	0.36	2.2	875	V stiff, Silty sand to clayey silt			20	2.06	0.72	05-09	06-10
11.5	5.8	6.7	0.22	1.9	1608	V stiff, Silty sand to clayey silt			10	1.02	0.44	00-02	00-02
12.0	4.4	5.1	0.12	2.6	1850	Stiff, Clayey silt to silty clay			18	0.41	0.24	00-02	00-02
12.5	4.3	4.9	0.13	2.6	1865	Soft, Silty clay to clay			18	0.39	0.25	00-02	00-02
13.0	4.5	5.1	0.18	3.8	1951	Soft, Silty clay to clay			18	0.41	0.35	00-02	00-02
13.5	4.3	4.8	0.15	3.4	2210	Soft, Silty clay to clay			18	0.38	0.30	00-02	00-02
14.0	4.0	4.5	0.15	3.6	2208	Soft, Silty clay to clay			18	0.35	0.30	00-02	00-02
14.5	4.3	4.8	0.19	4.7	2498	Soft, Clay			18	0.38	0.30	00-02	00-02
15.0	3.8	4.2	0.17	4.2	2463	Soft, Clay			18	0.32	0.35	00-02	00-02
15.5	4.3	4.8	0.19	4.5	2607	Soft, Clay			18	0.38	0.35	00-02	00-02
16.0	4.2	4.6	0.19	4.4	2577	Soft, Clay			18	0.36	0.38	00-02	00-02
16.5	4.5	4.9	0.21	4.7	2472	Soft, Clay			18	0.39	0.41	00-02	00-02
17.0	4.1	4.5	0.17	4.1	2459	Soft, Clay			18	0.34	0.34	00-02	00-02
17.5	4.0	4.3	0.17	4.1	2245	Soft, Clay			18	0.32	0.35	00-02	00-02
18.0	4.7	5.0	0.17	2.3	1578	Soft, Clayey silt to silty clay			18	0.40	0.34	00-02	00-02
18.5	7.5	8.1	0.24	3.3	1856	Stiff, Silty clay to clay			18	1.27	0.48	00-02	00-02
19.0	5.3	5.7	0.23	3.8	1811	Firm, Silty clay to clay			10	0.84	0.47	00-02	00-02
19.5	5.6	6.0	0.25	3.8	1473	Firm, Silty clay to clay			10	0.89	0.50	00-02	00-02
20.0	10.3	11.0	0.19	1.4	1089	Stiff, Silty sand to clayey silt			15	1.22	0.37	00-02	00-02
20.5	15.1	16.0	0.08	0.6	984	V loose, Silty sand to sandy silt	31-36	0-20				00-02	00-02
21.0	18.9	20.0	0.14	0.8	872	Loose, Silty sand to sandy silt	31-36	20-40				02-04	02-04
21.5	21.0	22.2	0.22	0.7	839	Loose, Silty sand to sandy silt	31-36	20-40				02-04	02-04
22.0	28.2	29.6	0.17	0.5	684	Loose, Silty sand to sandy silt	36-37	20-40				04-06	04-06
22.5	42.5	44.4	0.27	0.6	704	Loose, Sand to silty sand	37-40	20-40				06-10	06-10
23.0	44.8	46.7	0.29	0.4	764	Loose, Sand to silty sand	37-40	20-40				06-10	06-10
23.5	88.0	91.4	0.44	0.5	766	Med dense, Sand to silty sand	40-42	40-60				14-19	15-20
24.0	100.6	104.1	0.75	0.8	722	Med dense, Sand to silty sand	40-42	40-60				19-29	20-30
24.5	80.3	82.8	0.46	0.5	658	Med dense, Sand to silty sand	40-42	40-60				15-19	15-20
25.0	59.8	61.5	0.48	0.7	675	Loose, Sand to silty sand	37-40	20-40				10-15	10-15

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25.5	61.7	63.2	0.29	0.4	717	Loose, Sand to silty sand	40-42	20-40				06 - 10	06 - 10
26.0	108.4	110.7	0.56	0.5	736	Med dense, Sand to silty sand	40-42	40-60				20 - 29	20 - 30
26.5	83.7	83.7	0.35	0.3	806	Loose, Sand to silty sand	40-42	20-40				15 - 20	15 - 20
27.0	80.8	81.9	0.74	0.9	793	Med dense, Sand to silty sand	40-42	40-60				15 - 20	15 - 20
27.5	43.5	44.0	0.43	0.7	849	Loose, Sand to silty sand	37-40	20-40				06 - 10	06 - 10
28.0	57.5	57.9	0.35	0.4	859	Loose, Sand to silty sand	37-40	20-40				06 - 10	06 - 10
28.5	104.0	104.4	0.15	0.1	716	Med dense, Sand to silty sand	40-42	40-60				15 - 20	15 - 20
29.0	119.9	120.0	1.58	1.3	731	Dense, Sand to silty sand	40-42	60-80				30 - 40	30 - 40
29.5	79.2	79.1	0.84	0.8	665	Med dense, Sand to silty sand	40-42	40-60				15 - 20	15 - 20
30.0	84.6	83.9	0.54	0.6	750	Med dense, Sand to silty sand	40-42	40-60				15 - 20	15 - 20
30.5	91.4	90.3	1.02	1.2	732	Med dense, Sand to silty sand	40-42	40-60				15 - 20	15 - 20
31.0	61.3	60.4	0.61	0.8	717	Med dense, Sand to silty sand	37-40	40-60				20 - 30	20 - 30
31.5	55.2	54.2	0.48	0.9	781	Loose, Sand to silty sand	37-40	40-60				10 - 15	10 - 15
32.0	51.1	50.1	0.49	0.9	798	Loose, Silty sand to sandy silt	37-40	20-40				10 - 15	10 - 15
32.5	50.5	49.4	0.51	1.0	857	Loose, Silty sand to sandy silt	37-40	20-40				08 - 10	08 - 10
33.0	51.1	49.8	0.54	1.1	912	Med dense, Silty sand to sandy silt	36-37	40-60				10 - 15	10 - 15
34.0	49.2	47.8	0.50	1.0	965	Loose, Silty sand to sandy silt	36-37	20-40				06 - 10	06 - 10
34.5	52.0	50.4	0.50	0.9	1040	Loose, Sand to silty sand	37-40	20-40				06 - 10	06 - 10
35.0	56.7	54.8	0.51	0.8	1098	Loose, Sand to silty sand	37-40	20-40				08 - 10	08 - 10
35.5	71.0	68.4	0.59	0.8	1162	Med dense, Sand to silty sand	37-40	40-60				10 - 15	10 - 15
36.0	75.9	72.9	0.67	0.9	1220	Med dense, Sand to silty sand	37-40	40-60				13 - 20	13 - 20
36.5	78.1	74.8	0.72	0.9	1289	Med dense, Sand to silty sand	37-40	40-60				15 - 20	15 - 20
37.0	74.2	70.9	0.74	1.0	1387	Med dense, Sand to silty sand	37-40	40-60				15 - 20	15 - 20
37.5	65.3	62.2	0.69	1.0	1482	Med dense, Sand to silty sand	37-40	40-60				10 - 15	10 - 15
38.0	64.1	60.9	0.64	1.0	1553	Med dense, Sand to silty sand	37-40	40-60				10 - 15	10 - 15
38.5	60.2	57.1	0.60	1.0	1665	Med dense, Sand to silty sand	37-40	40-60				11 - 16	10 - 15
39.0	64.0	60.5	0.56	0.8	1588	Loose, Sand to silty sand	37-40	20-40				10 - 15	10 - 15
39.5	75.3	71.0	0.60	0.7	1646	Med dense, Sand to silty sand	40-42	40-60				11 - 16	10 - 15
40.0	90.5	85.1	0.63	0.6	1760	Med dense, Sand to silty sand	40-42	40-60				10 - 15	10 - 15
40.5	112.1	105.1	0.63	0.5	1743	Med dense, Sand to silty sand	40-42	40-60				15 - 20	15 - 20
41.0	131.2	122.8	0.75	0.6	1762	Med dense, Sand to silty sand	40-42	40-60				18 - 21	15 - 20
41.5	130.8	122.1	0.88	0.7	1802	Med dense, Sand to silty sand	40-42	40-60				21 - 32	20 - 30
42.0	131.3	122.3	0.72	0.5	1845	Med dense, Sand to silty sand	40-42	40-60				21 - 32	20 - 30
42.5	146.1	135.7	0.71	0.5	1891	Med dense, Sand to silty sand	40-42	40-60				21 - 32	20 - 30
43.0	133.7	123.9	1.12	0.8	1961	Med dense, Sand to silty sand	40-42	40-60				22 - 32	20 - 30
43.5	99.3	91.8	1.18	1.0	2025	Med dense, Sand to silty sand	40-42	40-60				22 - 32	20 - 30
44.0	104.7	96.6	0.82	0.6	2008	Med dense, Sand to silty sand	40-42	40-60				22 - 32	20 - 30
44.5	146.0	134.3	0.79	0.5	2019	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
45.0	155.8	143.0	1.01	0.7	2012	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
45.5	135.1	123.7	1.18	0.8	2116	Med dense, Sand to silty sand	40-42	40-60				33 - 44	30 - 40
46.0	140.9	126.7	0.94	0.7	2115	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
46.5	125.4	114.3	1.24	0.9	2103	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
47.0	109.4	99.5	1.09	0.9	2206	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
47.5	110.2	100.0	1.11	1.0	2224	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
48.0	108.9	98.6	1.00	0.9	2260	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
48.5	108.1	97.7	1.01	0.9	2334	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
49.0	108.9	98.2	1.01	0.9	2295	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
49.5	110.5	99.4	0.85	0.8	2351	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
50.0	121.5	109.0	0.99	0.8	2260	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30

Notes:

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50.5	130.8	117.1	0.6	2296	Med dense, Sand to silty sand	40-42	40-60				22-34	20-30
51.0	138.1	123.4	0.6	2430	Med dense, Sand to silty sand	40-42	40-60				22-34	20-30
51.5	145.8	130.0	0.5	2425	Med dense, Sand to silty sand	40-42	40-60				22-34	20-30
52.0	152.4	135.5	0.6	2508	Med dense, Sand to silty sand	40-42	40-60				22-34	20-30
52.5	145.8	129.4	0.7	2735	Med dense, Sand to silty sand	40-42	40-60				23-34	20-30
53.0	155.0	137.3	0.4	2774	Med dense, Sand to silty sand	42-46	40-60				23-34	20-30
53.5	160.4	141.8	0.6	2587	Med dense, Sand to silty sand	40-42	40-60				34-45	30-40
54.0	152.7	134.7	0.7	2571	Med dense, Sand to silty sand	40-42	40-60				23-34	20-30
54.5	140.8	124.0	0.4	2595	Med dense, Sand to silty sand	40-42	40-60				23-34	20-30
55.0	181.3	158.2	0.45	3020	Med dense, Sand to silty sand	42-46	60-80				23-34	20-30
55.5	208.8	183.1	1.31	2481	Dense, Sand to silty sand	42-46	60-80				46-68	40-60
56.0	151.1	132.2	1.29	2739	Med dense, Sand to silty sand	40-42	40-60				34-46	30-40
56.5	161.2	140.8	1.14	2722	Med dense, Sand to silty sand	40-42	40-60				34-46	30-40
57.0	209.4	182.5	0.96	2993	Med dense, Sand to silty sand	42-46	40-60				34-46	30-40
57.5	214.1	186.2	1.26	3230	Med dense, Sand to silty sand	42-46	40-60				46-69	40-60
58.0	192.6	167.2	1.04	2778	Med dense, Sand to silty sand	42-46	40-60				35-46	30-40
58.5	235.5	204.0	0.61	2747	Med dense, Sa gravel to gr sand	42-46	40-60				46-69	40-60
59.0	289.8	250.8	0.63	3127	Med dense, Sa gravel to gr sand	+46	40-60				46-69	40-60
59.5	332.2	286.6	1.00	3097	Dense, Sa gravel to gr sand	+46	60-80				46-70	40-60
60.0	394.1	339.4	1.18	3094	Dense, Sa gravel to gr sand	+46	60-80				70-115	60-99
60.5	327.8	281.8	3.33	3148	Dense, Sand to silty sand	42-46	60-80				47-70	40-60
61.0	201.2	172.6	2.10	2858	Dense, Sand to silty sand	42-46	60-80				23-35	20-30
61.5	146.3	125.3	1.39	2977	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
62.0	138.6	118.4	1.37	2927	Med dense, Sand to silty sand	40-42	40-60				35-47	30-40
62.5	144.7	123.5	1.56	2903	Med dense, Sand to silty sand	40-42	40-60				35-47	30-40
63.0	125.7	107.0	1.46	2953	Med dense, Sand to silty sand	40-42	40-60				47-71	40-60
63.5	99.9	84.9	1.31	2905	Med dense, Sand to silty sand	40-42	40-60				48-71	40-60
64.0	198.5	135.3	1.51	2840	Med dense, Sand to silty sand	37-40	40-60				48-72	40-60
64.5	162.4	137.5	1.69	2880	Med dense, Sand to silty sand	40-42	40-60				36-48	30-40
65.0	232.2	196.2	0.78	4457	Med dense, Sa gravel to gr sand	42-46	60-80				48-72	40-60
65.5	295.0	248.9	2.80	3293	Dense, Sand to silty sand	42-46	60-80				36-48	30-40
66.0	221.8	186.8	1.84	3129	Dense, Sand to silty sand	42-46	60-80				48-71	40-60
66.5	235.9	198.3	1.14	3163	Med dense, Sand to silty sand	42-46	60-80				48-71	40-60
67.0	188.6	158.3	1.81	3048	Dense, Sand to silty sand	42-46	60-80				48-72	40-60
67.5	178.9	148.2	1.50	3132	Med dense, Sand to silty sand	40-42	40-60				36-48	30-40
68.0	187.0	156.3	1.74	3174	Med dense, Sand to silty sand	42-46	40-60				48-72	40-60
68.5	208.6	174.1	2.11	3136	Dense, Sand to silty sand	42-46	60-80				36-48	30-40
69.0	187.8	156.3	3.13	3235	Dense, Sand to silty sand	40-42	60-80				48-72	40-60
69.5	137.6	114.4	2.49	3236	Dense, Silty sand to sandy silt	40-42	60-80				36-48	30-40
70.0	142.6	118.4	2.04	3194	Dense, Sand to silty sand	40-42	60-80				36-48	30-40
70.5	134.4	111.4	2.53	3345	Dense, Silty sand to sandy silt	37-40	60-80				36-48	30-40
71.0	148.6	123.0	2.19	3212	Dense, Sand to silty sand	40-42	60-80				36-48	30-40
71.5	167.1	138.0	2.25	3189	Dense, Sand to silty sand	40-42	60-80				48-73	40-60
72.0	180.1	148.5	2.09	3152	Dense, Sand to silty sand	40-42	60-80				48-73	40-60
72.5	179.3	147.6	2.08	3215	Dense, Sand to silty sand	40-42	60-80				48-73	40-60
73.0	200.4	164.7	1.66	3466	Med dense, Sand to silty sand	42-46	40-60				49-73	40-60
73.5	243.9	200.1	1.06	3581	Med dense, Sand to silty sand	42-46	40-60				49-73	40-60
74.0	167.0	136.7	1.70	3410	Med dense, Sand to silty sand	40-42	40-60				37-49	30-40
74.5	130.7	106.9	1.94	3604	Med dense, Sand to silty sand	40-42	40-60				24-37	20-30
75.0	118.6	98.8	1.85	3612	Dense, Silty sand to sandy silt	37-40	60-80				25-37	20-30

Notes:

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Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.

Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.

Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS
PROJECT NAME: Stratford Army Engine Plant
PROJECT NUMBER: 99-120-050
R2DATE: 5-10-1999 TIME: 13:53:20.59
SOUNDING NUMBER: CP-9912

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
75.5	112.8	91.9	1.70	1.5	3632	Med dense, Silty sand to sandy silt	37-40	40-60				25-37	20-30
76.0	117.7	95.7	1.66	1.4	3579	Med dense, Silty sand to sandy silt	37-40	40-60				25-37	20-30
76.5	126.2	102.5	1.73	1.3	3531	Med dense, Sand to silty sand	40-42	40-60				25-37	20-30
77.0	136.0	110.3	1.87	1.3	3485	Med dense, Sand to silty sand	40-42	40-60				37-49	30-40
77.5	147.3	119.3	2.40	1.6	3424	Dense, Silty sand to sandy silt	40-42	60-80				37-49	30-40
78.0	154.9	125.2	2.70	1.8	3565	Dense, Silty sand to sandy silt	37-40	60-80				49-74	40-60
78.5	152.2	122.9	2.28	1.5	3487	Dense, Sand to silty sand	40-42	60-80				37-50	30-40
79.0	149.3	120.3	2.21	1.5	3485	Dense, Sand to silty sand	40-42	60-80				37-50	30-40
79.5	140.8	113.3	1.98	1.4	3500	Dense, Sand to silty sand	40-42	60-80				37-50	30-40
80.0	140.6	113.0	1.86	1.3	3515	Med dense, Sand to silty sand	40-42	40-60				37-50	30-40
80.5	144.3	115.7	1.91	1.3	3512	Dense, Sand to silty sand	40-42	60-80				37-50	30-40
81.0	142.5	114.1	1.94	1.3	3511	Dense, Sand to silty sand	40-42	60-80				37-50	30-40
81.5	142.5	113.9	2.06	1.4	3617	Dense, Sand to silty sand	40-42	60-80				37-50	30-40
82.0	149.4	119.3	2.40	1.6	3692	Dense, Silty sand to sandy silt	37-40	60-80				38-50	30-40
82.5	144.1	114.8	2.23	1.5	3606	Dense, Silty sand to sandy silt	40-42	60-80				38-50	30-40
83.0	152.1	121.0	2.19	1.4	3613	Dense, Sand to silty sand	40-42	60-80				38-50	30-40
83.5	156.1	124.0	2.31	1.4	3677	Dense, Sand to silty sand	40-42	60-80				38-50	30-40
84.0	161.9	128.4	2.19	1.3	3530	Dense, Sand to silty sand	40-42	60-80				38-50	30-40
84.5	188.4	148.2	1.93	1.0	3591	Dense, Sand to silty sand	40-42	60-80				38-50	30-40
85.0	201.2	159.1	2.29	1.1	3706	Dense, Sand to silty sand	40-42	60-80				38-51	30-40
85.5	218.8	172.8	1.92	0.8	3630	Dense, Sand to silty sand	42-46	60-80				51-76	40-60
86.0	246.4	194.3	1.39	0.6	3609	Med dense, Sand to silty sand	42-46	40-60				51-76	40-60
86.5	254.7	200.5	1.87	0.7	3803	Dense, Sand to silty sand	42-46	60-80				51-76	40-60
87.0	233.3	183.4	2.09	0.9	3796	Dense, Sand to silty sand	42-46	60-80				51-76	40-60
87.5	203.2	159.5	1.88	0.8	3795	Med dense, Sand to silty sand	42-46	40-60				51-76	40-60
88.0	208.7	163.5	1.86	0.8	3869	Med dense, Sand to silty sand	42-46	40-60				38-51	30-40
88.5	180.1	141.0	1.72	0.9	3993	Med dense, Sand to silty sand	40-42	40-60				51-77	40-60
89.0	162.6	127.0	1.67	1.0	3860	Med dense, Sand to silty sand	40-42	40-60				38-51	30-40
89.5	153.1	119.4	1.64	1.0	3876	Med dense, Sand to silty sand	40-42	40-60				38-51	30-40
90.0	165.3	128.8	1.90	1.1	3932	Dense, Sand to silty sand	40-42	60-80				26-38	20-30
90.5	168.7	131.2	2.06	1.2	3929	Dense, Sand to silty sand	40-42	60-80				39-51	30-40
91.0	185.0	128.2	2.15	1.3	3922	Dense, Sand to silty sand	40-42	60-80				39-51	30-40
91.5	157.3	122.0	2.14	1.3	4019	Dense, Sand to silty sand	40-42	60-80				39-51	30-40
92.0	156.1	122.5	2.23	1.4	4059	Dense, Sand to silty sand	40-42	60-80				39-52	30-40
92.5	165.0	127.7	2.85	1.5	4098	Dense, Sand to silty sand	40-42	60-80				52-78	40-60
93.0	166.9	128.9	2.58	1.6	4038	Dense, Silty sand to sandy silt	40-42	60-80				52-78	40-60
93.5	164.0	126.5	3.30	2.0	4093	Dense, Silty sand to sandy silt	37-40	60-80				52-78	40-60
94.0	152.0	117.1	3.36	2.1	4291	Dense, Silty sand to sandy silt	37-40	60-80				52-78	40-60
94.5	153.7	118.2	3.44	2.2	4512	Dense, Silty sand to sandy silt	37-40	60-80				52-78	40-60
95.0	182.7	125.0	3.33	2.1	4476	Dense, Silty sand to sandy silt	37-40	60-80				52-78	40-60
95.5	161.8	124.1	3.27	2.0	4465	Dense, Silty sand to sandy silt	37-40	60-80				52-78	40-60
96.0	161.5	123.7	3.34	1.8	4356	Dense, Silty sand to sandy silt	37-40	60-80				52-78	40-60
96.5	188.8	144.4	4.20	2.3	4641	V dense, Silty sand to sandy silt	37-40	80-100				52-78	40-60
97.0	160.3	122.4	4.22	2.4	4579	V dense, Silty sand to sandy silt	37-40	80-100				52-78	40-60
97.5	148.6	113.3	3.84	2.5	4686	V dense, Silty sand to sandy silt	37-40	80-100				52-79	40-60
98.0	157.1	119.7	3.66	2.3	4646	V dense, Silty sand to sandy silt	37-40	80-100				53-79	40-60
98.5	157.8	120.0	3.99	2.5	4686	V dense, Silty sand to sandy silt	37-40	80-100				53-79	40-60
99.0	161.5	122.7	4.41	2.9	4520	V dense, Gr el sand to cl gr sand	36-37	80-100				79-130	60-99
99.5	140.0	106.2	4.30	2.8	4753	Hard, Sandy silt to sandy clay	37-40	80-100	30	8.94	8.61	53-79	40-60
100.0	155.8	118.0	3.69	2.1	4759	Dense, Silty sand to sandy silt	37-40	60-80				53-78	40-60

Notes:

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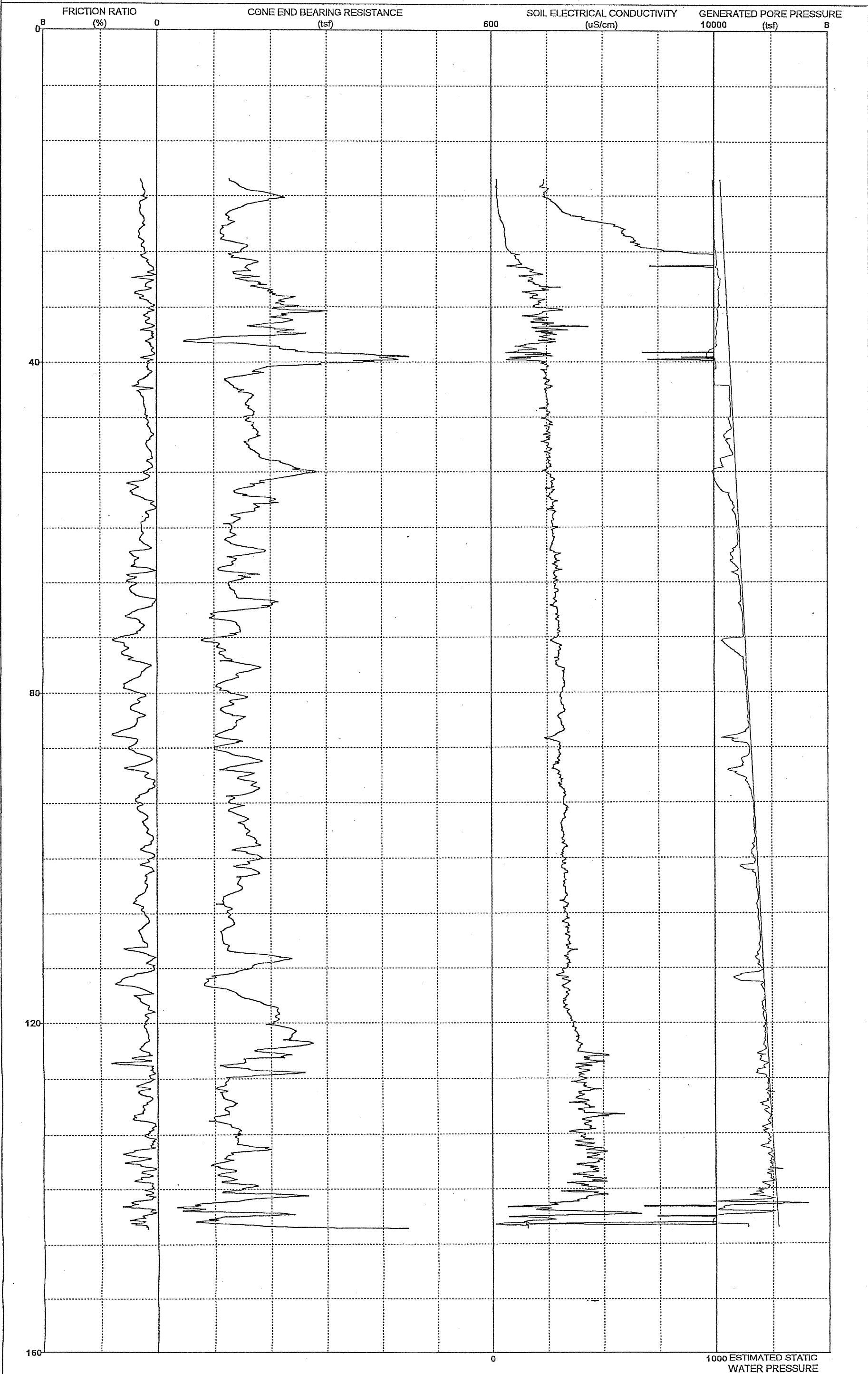
Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.

Both undrained and drained parameters can be estimated for these soils.

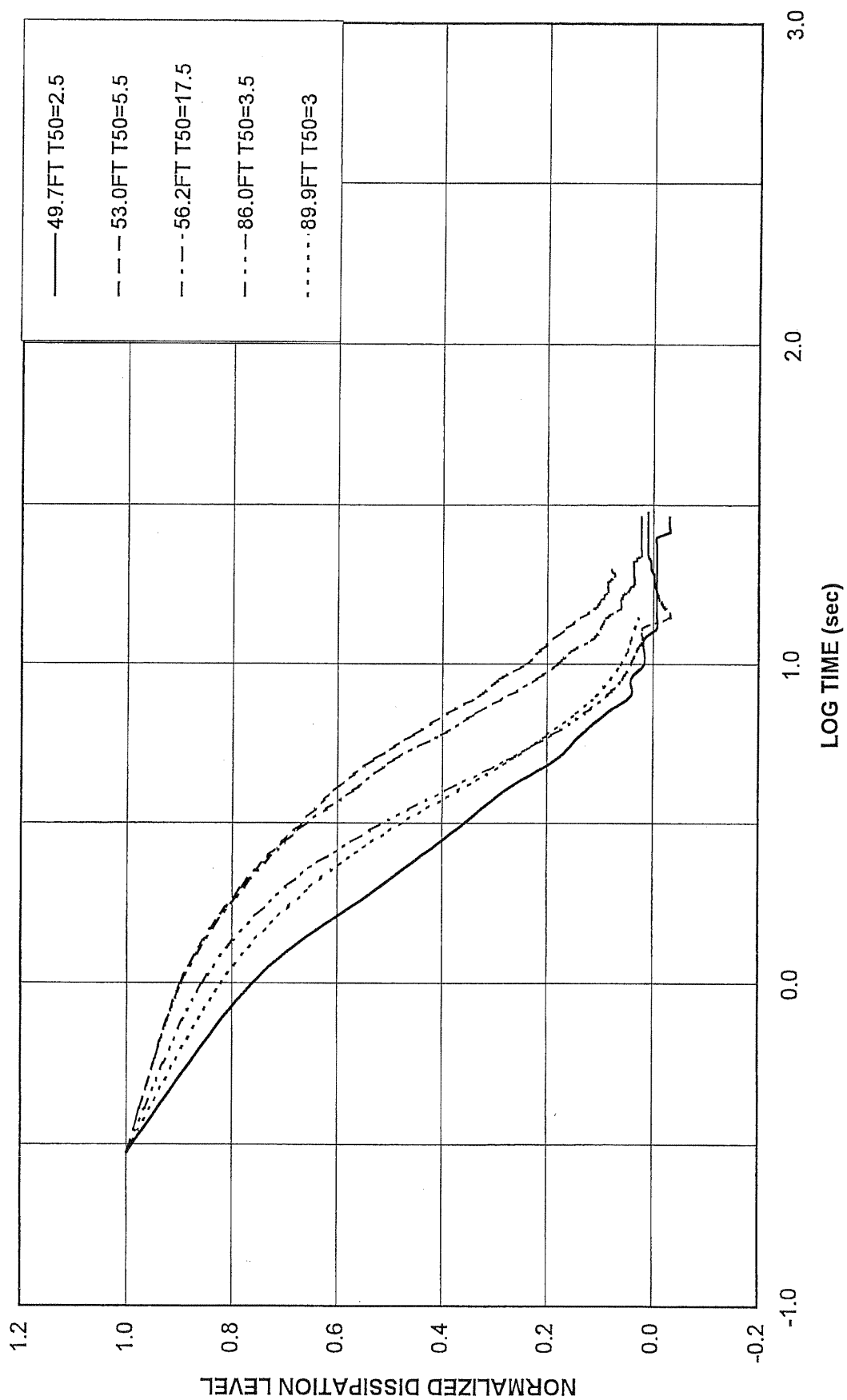
Structure rate of loading should be considered in choosing which strength parameters to use for design.

Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

CPTU-EC LOG



STRATIGRAPHICS
PORE WATER PRESSURE DISSIPATION TEST
Stratford Army Engine Plant CP-9913A



STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant
PROJECT NUMBER: 99-120-050
R2DATE: 5-12-1999 TIME: 14:46:19.41
SOUNDING NUMBER: CP-9913

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
1.0	Prepunched To 18.00'												
1.5	Prepunched To 18.00'												
2.0	Prepunched To 18.00'												
2.5	Prepunched To 18.00'												
3.0	Prepunched To 18.00'												
3.5	Prepunched To 18.00'												
4.0	Prepunched To 18.00'												
4.5	Prepunched To 18.00'												
5.0	Prepunched To 18.00'												
5.5	Prepunched To 18.00'												
6.0	Prepunched To 18.00'												
6.5	Prepunched To 18.00'												
7.0	Prepunched To 18.00'												
7.5	Prepunched To 18.00'												
8.0	Prepunched To 18.00'												
8.5	Prepunched To 18.00'												
9.0	Prepunched To 18.00'												
9.5	Prepunched To 18.00'												
10.0	Prepunched To 18.00'												
10.5	Prepunched To 18.00'												
11.0	Prepunched To 18.00'												
11.5	Prepunched To 18.00'												
12.0	Prepunched To 18.00'												
12.5	Prepunched To 18.00'												
13.0	Prepunched To 18.00'												
13.5	Prepunched To 18.00'												
14.0	Prepunched To 18.00'												
14.5	Prepunched To 18.00'												
15.0	Prepunched To 18.00'												
15.5	Prepunched To 18.00'												
16.0	Prepunched To 18.00'												
16.5	Prepunched To 18.00'												
17.0	Prepunched To 18.00'												
17.5	Prepunched To 18.00'												
18.0	Prepunched To 18.00'												
18.5	141.1	152.4	1.47	1.0	235	Dense, Sand to silty sand	40-42	60-80				37 - 56	40 - 60
19.0	151.5	162.9	1.59	1.0	247	Dense, Sand to silty sand	42-46	60-80				37 - 56	40 - 60
19.5	181.7	194.7	1.68	0.8	244	Dense, Sand to silty sand	42-46	60-80				37 - 56	40 - 60
20.0	213.7	228.0	2.12	1.0	241	Dense, Sand to silty sand	42-46	60-80				37 - 56	40 - 60
20.5	197.3	209.7	2.04	1.0	270	Dense, Sand to silty sand	42-46	60-80				38 - 56	40 - 60
21.0	163.3	172.9	2.17	1.2	291	Dense, Sand to silty sand	40-42	60-80				38 - 57	40 - 60
21.5	143.6	151.4	1.82	1.2	309	Dense, Sand to silty sand	40-42	60-80				38 - 57	40 - 60
22.0	126.3	132.7	1.58	1.2	328	Dense, Sand to silty sand	40-42	60-80				29 - 38	30 - 40
22.5	126.7	132.5	1.20	0.9	400	Med dense, Sand to silty sand	40-42	40-60				29 - 38	30 - 40
23.0	132.5	138.1	1.16	0.9	458	Med dense, Sand to silty sand	40-42	40-60				29 - 38	30 - 40
23.5	125.5	130.4	1.10	0.9	558	Med dense, Sand to silty sand	40-42	40-60				29 - 39	30 - 40
24.0	113.2	117.1	1.42	1.2	603	Med dense, Sand to silty sand	40-42	40-60				29 - 39	30 - 40
24.5	112.5	116.1	1.40	1.2	596	Med dense, Sand to silty sand	40-42	40-60				29 - 39	30 - 40
25.0	115.5	118.7	1.51	1.3	637	Dense, Sand to silty sand	40-42	60-80				29 - 39	30 - 40

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STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant

PROJECT NUMBER: 99-120-050

R2DATE: 5-12-1999 TIME: 14:46:19.41

SOUNDING NUMBER: CP-9913

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
25.5	133.3	136.6	1.35	0.9	633	Med dense, Sand to silty sand	40-42	40-60				29-39	30-40
26.0	160.6	163.9	1.70	1.1	663	Dense, Sand to silty sand	40-42	60-80				39-59	40-60
26.5	148.6	151.2	1.26	0.9	773	Med dense, Sand to silty sand	40-42	40-60				29-39	30-40
27.0	126.9	128.6	1.29	0.9	1206	Med dense, Sand to silty sand	40-42	40-60				30-39	30-40
27.5	145.4	146.9	1.35	0.8	1108	Med dense, Sand to silty sand	40-42	40-60				30-40	30-40
28.0	177.0	178.3	0.99	0.6	1307	Med dense, Sand to silty sand	42-46	40-60				40-60	40-60
28.5	157.4	158.1	0.46	0.3	1040	Med dense, Sand to silty sand	42-46	40-60				30-40	30-40
29.0	157.9	158.0	0.72	0.5	1813	Med dense, Sand to silty sand	42-46	40-60				30-40	30-40
29.5	149.2	148.8	0.23	0.3	1924	Med dense, Sand to silty sand	42-46	40-60				20-30	20-30
30.0	146.6	145.8	0.51	0.5	1870	Med dense, Sand to silty sand	42-46	40-60				30-40	30-40
30.5	180.7	179.2	1.40	0.8	1942	Dense, Sand to silty sand	42-46	60-80				30-40	30-40
31.0	179.9	177.9	0.69	0.4	2775	Med dense, Sand to silty sand	42-46	40-60				40-61	40-60
31.5	203.7	200.8	3.17	1.5	1481	Dense, Sand to silty sand	40-42	60-80				30-40	30-40
32.0	223.1	219.3	1.92	0.9	1883	Dense, Sand to silty sand	42-46	60-80				41-61	40-60
32.5	217.0	212.6	1.31	0.6	2307	Dense, Sand to silty sand	42-46	60-80				41-61	40-60
33.0	219.2	214.2	0.62	0.3	2151	Med dense, Sa gravel to gr sand	42-46	40-60				41-61	40-60
33.5	205.0	199.7	1.57	0.5	2657	Med dense, Sand to silty sand	42-46	40-60				41-62	40-60
34.0	262.2	254.7	1.93	0.7	2348	Dense, Sand to silty sand	42-46	60-80				41-62	40-60
34.5	206.9	200.4	1.59	0.7	1873	Dense, Sand to silty sand	42-46	60-80				41-62	40-60
35.0	236.7	228.6	1.73	0.8	2368	Dense, Sand to silty sand	42-46	60-80				41-62	40-60
35.5	176.6	170.1	0.45	0.2	2578	Med dense, Sand to silty sand	42-46	40-60				31-42	30-40
36.0	212.4	204.0	2.41	1.0	2772	Dense, Sand to silty sand	42-46	60-80				42-62	40-60
36.5	243.5	233.3	0.58	0.3	2303	Med dense, Sa gravel to gr sand	42-46	40-60				42-63	40-60
37.0	115.9	110.7	0.41	0.3	2281	Med dense, Sand to silty sand	37-40	20-40				21-31	20-30
37.5	47.9	45.6	1.24	0.7	2563	Loose, Sand to silty sand	42-46	40-60				06-10	06-10
38.0	180.9	171.9	0.88	0.4	1533	Med dense, Sand to silty sand	42-46	40-60				32-42	30-40
38.5	231.1	219.0	1.11	0.4	1960	Med dense, Sand to silty sand	42-46	40-60				42-63	40-60
39.0	324.3	306.6	0.53	0.1	2525	Dense, Sa gravel to gr sand	+46	80-100				42-63	40-60
39.5	411.1	367.7	3.61	0.9	1667	V dense, Sand to silty sand	42-46	60-80				64-105	60-99
40.0	362.5	341.0	1.79	0.5	2484	Dense, Sa gravel to gr sand	+46	60-80				43-64	40-60
40.5	201.3	188.9	1.90	0.7	2582	Dense, Sand to silty sand	42-46	40-60				32-43	30-40
41.0	175.0	163.7	0.94	0.5	2314	Med dense, Sand to silty sand	42-46	40-60				32-43	30-40
41.5	168.3	157.1	1.03	0.6	2465	Med dense, Sand to silty sand	40-42	40-60				21-32	20-30
42.0	122.3	113.9	1.40	1.0	2394	Med dense, Sand to silty sand	40-42	60-80				32-43	30-40
42.5	127.4	118.4	1.68	1.3	2501	Dense, Sand to silty sand	40-42	40-60				32-43	30-40
43.0	135.3	125.4	1.09	0.9	2699	Med dense, Sand to silty sand	40-42	40-60				32-43	30-40
43.5	146.0	135.0	2.01	1.3	2469	Dense, Sand to silty sand	40-42	60-80				32-43	30-40
44.0	170.4	157.2	1.81	1.1	2482	Dense, Sand to silty sand	40-42	60-80				43-65	40-60
44.5	161.6	148.7	1.49	0.9	2525	Med dense, Sand to silty sand	40-42	40-60				33-44	30-40
45.0	156.3	143.5	1.47	0.9	2565	Med dense, Sand to silty sand	40-42	40-60				33-44	30-40
45.5	160.9	147.3	1.43	0.8	2382	Med dense, Sand to silty sand	40-42	40-60				33-44	30-40
46.0	171.7	156.8	1.41	0.8	2580	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
46.5	162.1	147.7	1.10	0.7	2518	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
47.0	157.1	142.8	1.03	0.6	2565	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
47.5	170.1	154.3	1.16	0.7	2640	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
48.0	170.5	154.3	0.83	0.5	2595	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
48.5	178.1	161.7	0.69	0.4	2591	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
49.0	178.9	161.2	0.77	0.4	2646	Med dense, Sand to silty sand	42-46	40-60				33-44	30-40
49.5	161.7	145.4	1.27	0.7	2515	Med dense, Sand to silty sand	40-42	40-60				33-44	30-40
50.0	159.6	143.2	1.22	0.8	2590	Med dense, Sand to silty sand	40-42	40-60				33-45	30-40

Notes:

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Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.

Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.

Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS

PROJECT NAME:Stratford Army Engine Plant

PROJECT NUMBER:99-120-050

R2DATE: 5-12-1999 TIME:14:46:19.41

SOUNDING NUMBER:CP-9913

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
50.5	164.7	147.5	0.98	0.6	2683	Med dense, Sand to silty sand	42-46	40-60				34-45	30-40
51.0	176.0	157.2	0.66	0.4	2532	Med dense, Sand to silty sand	42-46	40-60				34-45	30-40
51.5	182.0	162.2	0.59	0.3	2648	Med dense, Sand to silty sand	42-46	40-60				34-45	30-40
52.0	214.5	190.8	1.49	0.7	2625	Dense, Sand to silty sand	42-46	60-80				45-67	40-60
52.5	234.5	208.1	0.81	0.3	2689	Med dense, Sand to silty sand	42-46	40-60				45-68	40-60
53.0	244.3	216.4	1.47	0.6	2541	Dense, Sand to silty sand	42-46	60-80				45-68	40-60
53.5	260.4	230.1	2.17	0.8	2552	Dense, Sand to silty sand	42-46	60-80				45-68	40-60
54.0	219.6	193.7	1.52	0.6	2722	Dense, Sand to silty sand	42-46	60-80				45-68	40-60
54.5	182.0	160.2	3.91	1.8	2707	Dense, Silty sand to sandy silt	40-42	60-80				45-68	40-60
55.0	170.8	150.1	2.24	1.3	2717	Dense, Sand to silty sand	40-42	60-80				45-68	40-60
55.5	138.9	121.8	2.77	1.8	2852	Dense, Silty sand to sandy silt	37-40	60-80				48-68	40-60
56.0	153.3	134.1	2.94	1.7	2842	Dense, Silty sand to sandy silt	40-42	60-80				48-69	40-60
56.5	205.6	179.5	1.89	0.9	2690	Dense, Sand to silty sand	42-46	60-80				48-69	40-60
57.0	212.2	184.9	0.97	0.4	2800	Med dense, Sand to silty sand	42-46	40-60				34-46	30-40
57.5	180.6	157.0	0.15	0.1	2788	Med dense, Sa gravel to gr sand	42-46	40-60				23-34	20-30
58.0	171.5	148.8	1.30	0.7	2806	Med dense, Sand to silty sand	40-42	40-60				35-46	30-40
58.5	131.9	114.3	0.85	0.6	2780	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
59.0	144.1	124.6	1.18	0.8	2832	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
59.5	119.3	103.0	1.52	1.1	2746	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
60.0	131.5	113.3	1.41	1.1	2983	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
60.5	129.1	111.0	1.33	1.0	2786	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
61.0	133.9	114.9	1.53	1.1	2761	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
61.5	122.1	104.5	1.45	1.1	2823	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
62.0	135.3	115.6	1.17	0.8	2745	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
62.5	174.9	149.2	0.83	0.5	2662	Med dense, Sand to silty sand	42-46	40-60				35-47	30-40
63.0	181.9	154.9	3.49	1.9	2823	Dense, Silty sand to sandy silt	40-42	60-80				47-70	40-60
63.5	125.2	106.4	2.31	1.4	2888	Dense, Sand to silty sand	37-40	60-80				35-47	30-40
64.0	135.5	115.0	2.15	1.6	2895	Dense, Silty sand to sandy silt	37-40	60-80				35-47	30-40
64.5	132.2	111.9	2.23	1.7	2820	Dense, Silty sand to sandy silt	37-40	60-80				35-47	30-40
65.0	108.2	91.5	0.35	0.3	2863	Med dense, Sand to silty sand	40-42	40-60				18-24	15-20
65.5	161.5	136.2	2.28	1.4	2867	Dense, Sand to silty sand	40-42	60-80				47-71	40-60
66.0	161.0	135.6	2.48	1.5	2821	Dense, Sand to silty sand	40-42	60-80				47-71	40-60
66.5	131.8	110.8	3.07	2.0	2943	Dense, Silty sand to sandy silt	37-40	60-80				48-71	40-60
67.0	127.6	107.1	2.23	1.7	2905	Dense, Silty sand to sandy silt	37-40	60-80				48-71	40-60
67.5	132.9	111.3	2.58	1.9	2930	Dense, Silty sand to sandy silt	37-40	60-80				48-72	40-60
68.0	138.5	115.8	2.20	1.6	2883	Dense, Silty sand to sandy silt	37-40	60-80				48-72	40-60
68.5	143.3	119.6	0.69	0.4	2876	Med dense, Sand to silty sand	40-42	40-60				24-36	20-30
69.0	213.1	177.5	0.44	0.2	2927	Med dense, Sa gravel to gr sand	42-46	40-60				36-48	30-40
69.5	189.2	157.3	0.82	0.4	2890	Med dense, Sand to silty sand	42-46	40-60				36-48	30-40
70.0	145.0	120.4	2.53	1.4	2872	Dense, Sand to silty sand	40-42	60-80				36-48	30-40
70.5	95.3	79.0	2.25	2.0	2963	Dense, Silty sand to sandy silt	36-37	60-80				24-36	20-30
71.0	95.4	78.9	1.85	1.6	3023	Med dense, Silty sand to sandy silt	37-40	40-60				24-36	20-30
71.5	138.5	114.4	1.67	1.2	3055	Med dense, Sand to silty sand	40-42	40-60				36-48	30-40
72.0	145.1	119.7	1.41	1.0	3032	Med dense, Sand to silty sand	40-42	40-60				36-48	30-40
72.5	147.0	121.0	1.87	1.3	2980	Dense, Sand to silty sand	40-42	60-80				24-36	20-30
73.0	136.9	112.5	2.77	2.0	3007	Dense, Silty sand to sandy silt	37-40	60-80				36-49	30-40
73.5	83.4	68.4	3.35	3.0	2817	Hard, Sandy silt to sandy clay	36-37	60-80	25	6.32	6.70	49-73	40-60
74.0	107.2	87.8	2.47	2.3	2854	Dense, Silty sand to sandy silt	37-40	60-80				37-49	30-40
74.5	105.8	86.5	2.31	2.0	3059	Dense, Silty sand to sandy silt	37-40	60-80				37-49	30-40
75.0	115.1	93.9	2.78	2.4	2998	Dense, Silty sand to sandy silt	37-40	60-80				49-74	40-60

Notes:

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STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant

PROJECT NUMBER: 99-120-050

R2DATE: 5-12-1999 TIME: 14:46:19.41

SOUNDING NUMBER: CP-9913

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
75.5	116.3	94.8	2.84	2.1	3022	Dense, Silty sand to sandy silt	37.40	60-80				37-49	30-40
76.0	131.7	107.1	2.49	1.9	3032	Dense, Silty sand to sandy silt	37.40	60-80				37-49	30-40
76.5	153.1	124.4	1.07	0.6	2929	Med dense, Sand to silty sand	40.42	40-60				25-37	20-30
77.0	180.7	146.6	1.43	0.8	3202	Med dense, Sand to silty sand	40.42	40-60				37-49	30-40
77.5	153.0	123.9	1.91	1.1	3222	Med dense, Sand to silty sand	40.42	40-60				37-49	30-40
78.0	142.3	115.1	1.81	1.3	3241	Med dense, Sand to silty sand	40.42	40-60				37-49	30-40
78.5	124.7	100.6	2.57	1.9	3248	Dense, Silty sand to sandy silt	37.40	60-80				37-50	30-40
79.0	105.6	85.1	2.66	2.3	3279	Dense, Silty sand to sandy silt	36.37	60-80				37-50	30-40
79.5	112.2	90.2	2.44	2.1	3197	Dense, Silty sand to sandy silt	37.40	60-80				37-50	30-40
80.0	129.3	103.9	1.73	1.2	3155	Med dense, Sand to silty sand	40.42	40-60				25-37	20-30
80.5	158.9	127.4	1.48	1.0	3100	Med dense, Sand to silty sand	40.42	40-60				37-50	30-40
81.0	128.2	102.7	1.78	1.2	3129	Med dense, Sand to silty sand	40.42	40-60				25-37	20-30
81.5	130.7	104.5	2.33	1.8	3187	Dense, Silty sand to sandy silt	37.40	60-80				37-50	30-40
82.0	111.5	89.0	2.13	1.8	3283	Dense, Silty sand to sandy silt	37.40	60-80				38-50	30-40
82.5	122.1	97.3	1.70	1.1	3170	Med dense, Sand to silty sand	40.42	40-60				25-38	20-30
83.0	152.7	121.5	1.90	1.3	3004	Dense, Sand to silty sand	40.42	40-60				25-38	20-30
83.5	139.7	111.0	2.04	1.4	3121	Dense, Sand to silty sand	40.42	60-80				38-50	30-40
84.0	139.2	110.5	2.21	1.6	3168	Dense, Silty sand to sandy silt	40.42	60-80				38-50	30-40
84.5	128.1	101.5	3.16	2.3	3277	Dense, Silty sand to sandy silt	37.40	60-80				51-78	40-60
85.0	107.5	85.0	3.66	3.1	2984	Hard, Sandy silt to sandy clay	37.40	60-80	30	6.82	7.33	51-78	40-60
85.5	117.0	92.4	2.17	1.6	2559	Dense, Silty sand to sandy silt	37.40	60-80				51-78	40-60
86.0	141.3	111.4	1.96	1.4	3026	Dense, Sand to silty sand	40.42	60-80				38-51	30-40
86.5	106.5	83.8	2.40	1.9	3109	Dense, Silty sand to sandy silt	37.40	60-80				25-38	20-30
87.0	112.1	88.1	2.01	1.6	3003	Dense, Silty sand to sandy silt	37.40	60-80				25-38	20-30
87.5	134.7	105.7	1.40	0.9	3114	Med dense, Sand to silty sand	40.42	40-60				25-38	20-30
88.0	173.0	135.6	0.62	0.4	2973	Med dense, Sand to silty sand	42.46	40-60				26-39	20-30
88.5	171.2	134.0	1.65	0.9	2894	Med dense, Sand to silty sand	40.42	40-60				38-51	30-40
89.0	127.5	99.6	3.15	2.1	2768	Dense, Silty sand to sandy silt	37.40	60-80				38-51	30-40
89.5	145.4	113.4	1.29	0.8	3015	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
90.0	157.0	122.4	1.43	0.9	3096	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
90.5	148.9	115.8	0.33	0.2	3194	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
91.0	170.5	132.4	0.50	0.3	3037	Med dense, Sand to silty sand	42.46	40-60				26-39	20-30
91.5	178.9	139.6	0.67	0.4	3157	Med dense, Sand to silty sand	42.46	40-60				26-39	20-30
92.0	168.8	130.8	1.29	0.7	3278	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
92.5	122.8	95.0	1.87	1.2	3330	Med dense, Sand to silty sand	37.40	40-60				26-39	20-30
93.0	128.1	98.9	2.02	1.5	3224	Dense, Silty sand to sandy silt	37.40	60-80				26-39	20-30
93.5	143.6	110.8	1.46	1.0	3232	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
94.0	131.6	101.4	2.02	1.4	3380	Med dense, Sand to silty sand	37.40	40-60				26-39	20-30
94.5	134.6	103.5	1.35	1.0	3315	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
95.0	153.3	117.8	0.99	0.6	3290	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
95.5	154.7	118.7	1.63	1.0	3191	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
96.0	153.8	117.8	1.33	0.8	3142	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
96.5	159.5	122.0	1.02	0.6	3170	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
97.0	158.1	120.8	0.98	0.6	3235	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
97.5	171.3	130.6	0.47	0.3	3215	Med dense, Sand to silty sand	42.46	40-60				26-39	20-30
98.0	174.8	133.1	0.93	0.4	3288	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
98.5	177.6	135.1	1.65	0.9	3357	Med dense, Sand to silty sand	40.42	40-60				39-53	30-40
99.0	133.4	101.3	1.56	1.0	3213	Med dense, Sand to silty sand	40.42	40-60				26-39	20-30
99.5	165.6	125.6	0.34	0.2	3181	Med dense, Sand to silty sand	42.46	40-60				26-40	20-30
100.0	184.2	139.5	0.96	0.5	3244	Med dense, Sand to silty sand	40.42	40-60				26-40	20-30

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STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant

PROJECT NUMBER: 99-120-050

R2DATE: 5-12-1999 TIME: 14:46:19.41

SOUNDING NUMBER: CP-9913

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
100.5	160.2	121.2	1.12	0.7	3159	Med dense, Sand to silty sand	40-42	40-60				26-40	20-30
101.0	136.2	102.9	0.94	0.6	3145	Med dense, Sand to silty sand	40-42	40-60				26-40	20-30
101.5	172.4	130.1	0.48	0.3	3301	Med dense, Sand to silty sand	42-46	40-60				20-30	20-30
102.0	173.6	130.8	1.63	0.8	3262	Med dense, Sand to silty sand	40-42	40-60				40-53	30-40
102.5	146.0	108.9	1.16	0.8	3276	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
103.0	141.6	106.4	1.22	0.8	3241	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
103.5	149.9	112.5	1.15	0.8	3251	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
104.0	141.5	106.0	1.27	0.9	3310	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
104.5	121.8	91.2	1.72	1.3	3399	Med dense, Sand to silty sand	37-40	40-60				27-40	20-30
105.0	116.0	86.7	1.79	1.5	3259	Med dense, Silty sand to sandy silt	37-40	40-60				27-40	20-30
105.5	108.7	81.1	1.76	1.5	3395	Med dense, Silty sand to sandy silt	37-40	40-60				27-40	20-30
106.0	130.0	96.9	1.26	1.0	3239	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
106.5	123.1	91.7	1.13	0.9	3288	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
107.0	125.9	93.6	0.94	0.7	3368	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
107.5	133.8	99.3	0.77	0.6	3278	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
108.0	134.9	100.0	1.11	0.8	3430	Med dense, Sand to silty sand	40-42	40-60				27-40	20-30
108.5	118.6	87.9	1.39	1.2	3449	Med dense, Sand to silty sand	37-40	40-60				27-40	20-30
109.0	112.8	83.5	1.39	1.2	3466	Med dense, Sand to silty sand	37-40	40-60				27-40	20-30
109.5	114.4	84.5	1.23	1.1	3514	Med dense, Sand to silty sand	37-40	40-60				27-40	20-30
110.0	115.5	85.2	1.09	0.9	3454	Med dense, Sand to silty sand	37-40	40-60				27-40	20-30
110.5	126.0	92.8	0.86	0.7	3357	Med dense, Sand to silty sand	40-42	40-60				20-27	15-20
111.0	123.5	90.9	2.95	2.1	3457	Dense, Silty sand to sandy silt	37-40	60-80				20-27	15-20
111.5	165.3	121.5	1.40	0.7	3407	Med dense, Sand to silty sand	40-42	40-60				41-54	30-40
112.0	216.1	158.6	0.37	0.2	3371	Med dense, Sand to silty sand	42-46	40-60				27-41	20-30
112.5	214.5	157.3	1.34	0.6	3503	Med dense, Sand to silty sand	42-46	40-60				27-41	20-30
113.0	168.8	123.6	0.46	0.2	3430	Med dense, Sand to silty sand	40-42	40-60				41-55	30-40
113.5	154.0	112.6	0.32	0.2	3211	Med dense, Sand to silty sand	40-42	40-60				27-41	20-30
114.0	127.3	93.0	2.21	1.5	3151	Med dense, Silty sand to sandy silt	37-40	40-60				27-41	20-30
114.5	100.3	73.2	2.45	2.3	3412	Dense, Silty sand to sandy silt	36-37	60-80	25	6.38		27-41	20-30
115.0	86.7	63.1	2.43	2.7	3506	Hard, Sandy silt to sandy clay					4.86	27-41	20-30
115.5	87.2	63.4	1.88	1.7	3163	Med dense, Silty sand to sandy silt	36-37	40-60				21-27	15-20
116.0	127.3	92.5	1.05	0.8	3457	Med dense, Sand to silty sand	40-42	40-60				28-41	20-30
116.5	141.4	102.7	0.86	0.8	3436	Med dense, Sand to silty sand	40-42	40-60				28-41	20-30
117.0	156.9	113.7	2.11	1.2	3301	Med dense, Sand to silty sand	40-42	40-60				41-55	30-40
117.5	183.4	132.8	1.96	1.0	3208	Med dense, Sand to silty sand	40-42	40-60				41-55	30-40
118.0	205.9	149.0	1.24	0.6	3281	Med dense, Sand to silty sand	42-46	40-60				41-55	30-40
118.5	211.4	152.7	0.33	0.3	3411	Med dense, Sand to silty sand	42-46	40-60				41-55	30-40
119.0	211.6	152.7	1.90	0.9	3474	Med dense, Sand to silty sand	42-46	40-60				41-55	30-40
119.5	214.2	154.3	1.37	0.7	3538	Med dense, Sand to silty sand	42-46	40-60				42-55	30-40
120.0	205.3	147.8	1.70	0.9	3702	Med dense, Sand to silty sand	42-46	40-60				42-55	30-40
120.5	224.5	161.4	2.19	0.9	3656	Dense, Sand to silty sand	42-46	60-80				56-83	40-60
121.0	244.2	175.3	1.88	0.8	3726	Dense, Sand to silty sand	42-46	60-80				56-83	40-60
121.5	237.0	170.0	1.55	0.6	3680	Med dense, Sand to silty sand	42-46	60-80				56-84	40-60
122.0	222.8	159.6	1.47	0.6	3669	Med dense, Sand to silty sand	42-46	60-80				42-56	30-40
122.5	270.0	193.1	2.55	1.0	4020	Dense, Sand to silty sand	42-46	60-80				42-56	30-40
123.0	203.0	145.0	3.01	1.2	3835	Dense, Sand to silty sand	42-46	60-80				56-84	40-60
123.5	193.6	136.2	1.13	0.6	4066	Med dense, Sand to silty sand	40-42	60-80				28-42	20-30
124.0	217.9	155.4	4.00	1.5	4485	Dense, Sand to silty sand	40-42	60-80				56-84	40-60
124.5	154.2	109.8	1.60	1.2	4463	Med dense, Sand to silty sand	40-42	40-60				28-42	20-30
125.0	111.8	79.5	0.48	0.6	4233	Med dense, Sand to silty sand	40-42	40-60				21-28	15-20

Notes:

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STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant

PROJECT NUMBER: 99-120-050

R2DATE: 5-12-1999 TIME: 14:46:19.41

SOUNDING NUMBER: CP-9913

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	SPT (N1)
125.5	162.5	115.5	1.14	0.5	3972	Med dense, Sand to silty sand	40-42	40-60				28-42	20-30
126.0	246.0	174.5	0.64	0.3	3903	Med dense, Sand to silty sand	42-46	40-60				42-56	30-40
126.5	122.5	86.8	1.06	0.6	4016	Med dense, Sand to silty sand	40-42	40-60				21-28	15-20
127.0	121.4	86.0	0.63	0.6	3879	Med dense, Sand to silty sand	40-42	40-60				21-28	15-20
127.5	119.2	84.3	1.56	1.3	4114	Med dense, Silty sand to sandy silt	37-40	40-60				28-42	20-30
128.0	103.3	72.9	0.55	0.6	4463	Loose, Sand to silty sand	40-42	20-40				14-21	10-15
128.5	110.9	78.2	1.57	1.4	3945	Med dense, Silty sand to sandy silt	37-40	40-60				28-43	20-30
129.0	114.3	80.5	0.55	0.5	3934	Med dense, Sand to silty sand	40-42	40-60				21-28	15-20
129.5	132.0	92.9	0.81	0.7	4340	Med dense, Sand to silty sand	40-42	40-60				21-28	15-20
130.0	133.6	93.9	1.51	1.0	4250	Med dense, Sand to silty sand	40-42	40-60				28-43	20-30
130.5	116.9	82.1	1.20	1.0	3930	Med dense, Sand to silty sand	37-40	40-60				21-28	15-20
131.0	124.5	87.3	1.89	1.4	4583	Med dense, Silty sand to sandy silt	37-40	40-60				21-28	15-20
131.5	89.5	69.7	2.20	1.8	4420	Med dense, Silty sand to sandy silt	36-37	40-60				29-43	20-30
132.0	116.7	81.7	0.29	0.3	4376	Loose, Sand to silty sand	40-42	20-40				14-21	10-15
132.5	132.3	92.5	0.72	0.4	4459	Med dense, Sand to silty sand	40-42	40-60				21-29	15-20
133.0	141.6	98.9	0.88	0.6	3685	Med dense, Sand to silty sand	40-42	40-60				29-43	20-30
133.5	137.9	96.2	1.20	0.9	4142	Med dense, Sand to silty sand	40-42	40-60				29-43	20-30
134.0	140.2	97.7	0.77	0.3	4516	Med dense, Sand to silty sand	40-42	40-60				22-29	15-20
134.5	140.9	98.1	0.61	0.4	4483	Med dense, Sand to silty sand	40-42	40-60				22-29	15-20
135.0	191.1	132.8	0.31	0.3	4474	Med dense, Sand to silty sand	42-46	40-60				29-43	20-30
135.5	143.3	99.5	3.11	1.8	5105	Dense, Silty sand to sandy silt	37-40	60-80				43-58	30-40
136.0	121.2	84.0	2.04	1.6	4684	Med dense, Silty sand to sandy silt	37-40	40-60				29-43	20-30
136.5	130.5	90.5	1.58	1.2	4932	Med dense, Sand to silty sand	37-40	40-60				22-29	15-20
137.0	96.1	66.5	1.25	1.3	3969	Med dense, Silty sand to sandy silt	37-40	40-60				22-29	15-20
137.5	112.7	77.9	0.16	0.1	4495	Loose, Sand to silty sand	40-42	20-40				14-22	10-15
138.0	124.1	85.7	1.51	1.1	4616	Med dense, Sand to silty sand	37-40	40-60				29-43	20-30
138.5	120.9	83.4	0.89	0.7	4485	Med dense, Sand to silty sand	40-42	40-60				22-29	15-20
139.0	126.6	86.6	2.22	1.6	4583	Dense, Silty sand to sandy silt	37-40	60-80				29-44	20-30
139.5	170.6	117.5	0.49	0.4	4132	Med dense, Sand to silty sand	40-42	40-60				29-44	20-30
140.0	150.1	103.2	1.26	0.8	4953	Med dense, Sand to silty sand	40-42	40-60				29-44	20-30
140.5	152.4	104.7	0.50	0.4	4651	Med dense, Sand to silty sand	40-42	40-60				29-44	20-30
141.0	210.6	144.5	0.82	0.4	4230	Med dense, Sand to silty sand	40-42	40-60				29-44	20-30
141.5	116.4	79.8	3.09	1.9	3860	Dense, Silty sand to sandy silt	42-46	40-60				29-44	20-30
142.0	61.1	41.8	1.55	2.4	2248	Hard, Sandy silt to sandy clay	37-40	60-80				29-44	20-30
142.5	51.0	34.9	1.08	0.5	2059	Loose, Sand to silty sand	36-37	20-40	25	4.20	3.09	15-22	10-15
143.0	239.3	163.5	0.78	0.8	6588	Med dense, Sand to silty sand	42-46	40-60				06-09	04-06
143.5	89.4	61.0	2.69	1.7	2475	Med dense, Silty sand to sandy silt	36-37	40-60				59-88	40-60
144.0	97.5	66.5	1.76	1.8	1392	Med dense, Silty sand to sandy silt	36-37	40-60				22-29	15-20
144.5	221.6	150.9	4.45	0.7	1570	Med dense, Sand to silty sand	42-46	40-60				29-44	20-30
												44-59	30-40

Notes:

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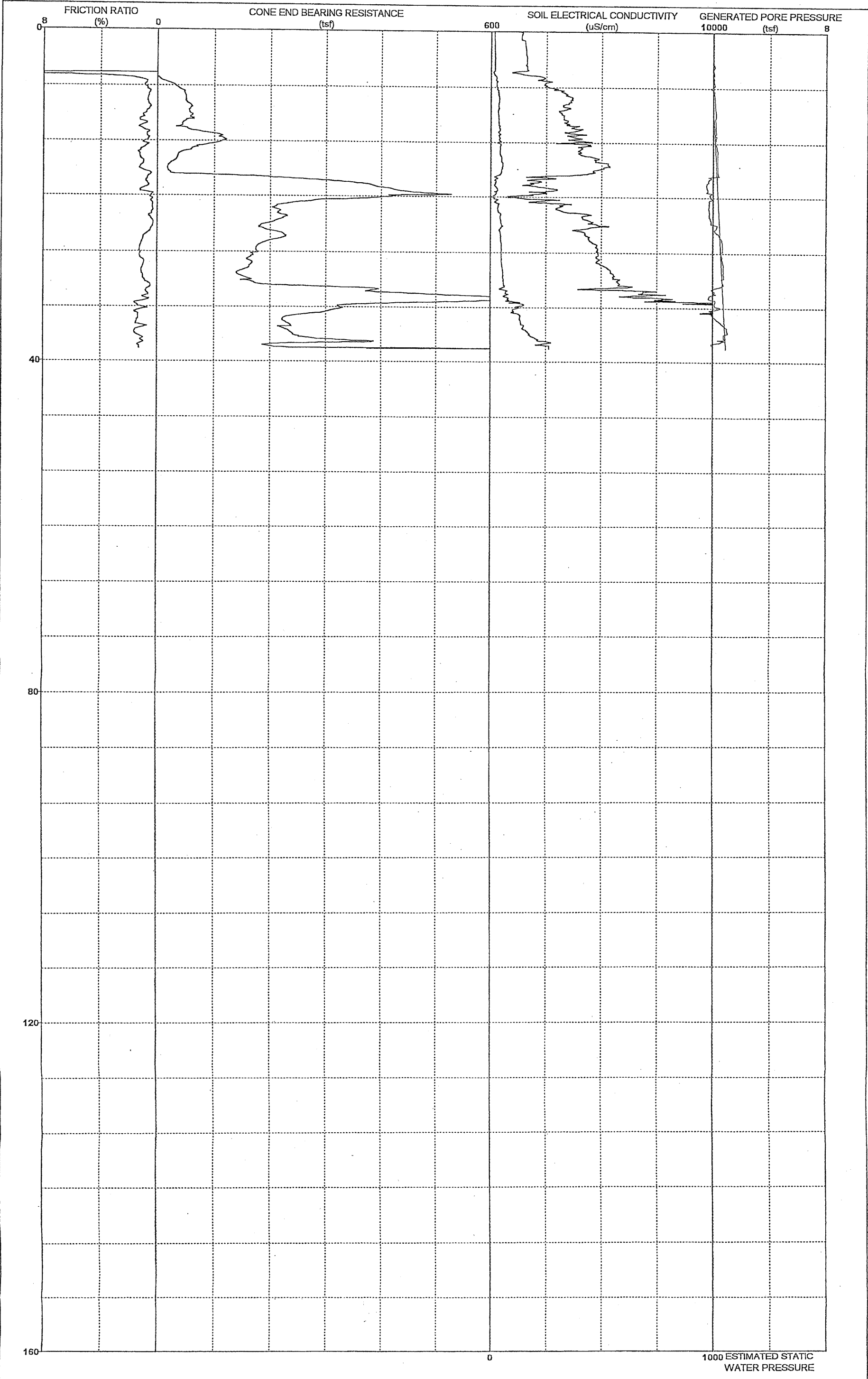
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CPTU-EC LOG



STRATIGRAPHICS

PROJECT NAME:Stratford Army Engine Plant

PROJECT NUMBER:99-120-050

R2DATE: 5-18-1999 TIME:16:50:05.11

SOUNDING NUMBER:CP-9914A

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (NT)
1.0	Prepunched To 25.00'												
1.5	Prepunched To 25.00'												
2.0	Prepunched To 25.00'												
2.5	Prepunched To 25.00'												
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24.5	Prepunched To 25.00'												
25.0	Prepunched To 25.00'												

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STRATIGRAPHICS

PROJECT NAME:Stratford Army Engine Plant
PROJECT NUMBER:99-120-050
R2DATE: 5-18-1999 TIME:16:50:05.11
SOUNDING NUMBER:CP-9914A

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
25.5	190.4	188.4	2.25	1.1	464	Dense, Sand to silty sand	42.46	80-80				40 - 61	40 - 60
26.0	177.0	174.6	2.20	1.2	477	Dense, Sand to silty sand	40.42	80-80				41 - 61	40 - 60
26.5	175.4	172.5	2.20	1.3	480	Dense, Sand to silty sand	40.42	80-80				41 - 61	40 - 60
27.0	169.6	166.4	2.16	1.2	486	Dense, Sand to silty sand	40.42	80-80				41 - 61	40 - 60
27.5	159.6	156.1	1.88	1.1	492	Dense, Sand to silty sand	40.42	80-80				41 - 61	40 - 60
28.0	168.1	163.9	1.56	0.9	493	Dense, Sand to silty sand	42.46	80-80				41 - 62	40 - 60
28.5	167.5	158.0	1.71	1.0	526	Dense, Sand to silty sand	40.42	80-80				41 - 62	40 - 60
29.0	146.4	141.9	1.74	1.1	542	Dense, Sand to silty sand	40.42	80-80				31 - 41	30 - 40
29.5	145.0	140.1	1.57	1.0	558	Med dense, Sand to silty sand	40.42	80-80				31 - 41	30 - 40
30.0	164.9	156.9	1.52	1.0	580	Dense, Sand to silty sand	40.42	80-80				41 - 62	40 - 60
30.5	174.2	167.5	1.44	0.6	581	Med dense, Sand to silty sand	42.46	80-80				31 - 42	30 - 40
31.0	366.8	351.7	2.14	0.6	458	Dense, Sa gravel to gr sand	+46	80-80				63 - 103	60 - 99
31.5	394.3	377.1	4.05	0.8	687	Dense, Sand to silty sand	42.46	80-80				63 - 104	60 - 99
32.0	615.2	586.7	5.36	0.8	605	V dense, Sa gravel to gr sand	+46	80-100				+ 105	+ 100
32.5	600.4	571.1	4.65	0.7	748	V dense, Sa gravel to gr sand	+46	80-100				+ 105	+ 100
33.0	351.3	333.2	6.88	1.5	1502	V dense, Sa gravel to si gr sand	42.46	80-100				+ 105	+ 100
33.5	326.0	308.5	2.34	0.7	1111	Dense, Sand to silty sand	42.48	80-80				63 - 105	60 - 99
34.0	289.0	272.8	4.81	1.5	1103	V dense, Sand to silty sand	42.46	80-100				+ 106	+ 100
34.5	228.2	214.8	3.39	1.3	1325	Dense, Sand to silty sand	42.46	80-80				42 - 64	40 - 60
35.0	224.5	210.8	3.32	1.4	1348	Dense, Sand to silty sand	42.46	80-80				64 - 105	60 - 99
35.5	237.5	222.4	1.53	1.0	1498	Dense, Sand to silty sand	42.46	80-80				43 - 64	40 - 60
36.0	228.6	213.6	3.56	1.5	1537	Dense, Sand to silty sand	40.42	80-80				64 - 106	60 - 99
36.5	239.2	222.9	3.92	1.6	1747	V dense, Sand to silty sand	40.42	80-100				64 - 106	60 - 99
37.0	260.5	242.2	3.41	1.1	1983	Dense, Sand to silty sand	42.46	80-80				65 - 106	60 - 99
37.5	366.5	358.4	3.22	1.0	2590	V dense, Sand to silty sand	42.48	80-100				+ 108	+ 100
38.0	187.5	173.4	4.55	1.4	2497	Dense, Sand to silty sand	40.42	80-80				43 - 65	40 - 60

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STRATIGRAPHICS

PROJECT NAME:Stratford Army Engine Plant

PROJECT NUMBER:99-120-050

R2DATE: 5-20-1999 TIME:10:55:48.01

SOUNDING NUMBER:cp-9915

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged		Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained		NORM SPT (N1')
				Friction (tsf)	Ratio (%)						Shear Strength (ksf)	Large Strain Shear Strength (ksf)	
1.0	Prepunched To 37.00'												
1.5	Prepunched To 37.00'												
2.0	Prepunched To 37.00'												
2.5	Prepunched To 37.00'												
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24.5	Prepunched To 37.00'												
25.0	Prepunched To 37.00'												

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STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant
PROJECT NUMBER: 99-120-050
R2DATE: 5-20-1999 TIME: 10:55:48.01
SOUNDING NUMBER: cp-9915

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
25.5	Prepunched To 37.00'												
26.0	Prepunched To 37.00'												
26.5	Prepunched To 37.00'												
27.0	Prepunched To 37.00'												
27.5	Prepunched To 37.00'												
28.0	Prepunched To 37.00'												
28.5	Prepunched To 37.00'												
29.0	Prepunched To 37.00'												
29.5	Prepunched To 37.00'												
30.0	Prepunched To 37.00'												
30.5	Prepunched To 37.00'												
31.0	Prepunched To 37.00'												
31.5	Prepunched To 37.00'												
32.0	Prepunched To 37.00'												
32.5	Prepunched To 37.00'												
33.0	Prepunched To 37.00'												
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34.0	Prepunched To 37.00'												
34.5	Prepunched To 37.00'												
35.0	Prepunched To 37.00'												
35.5	Prepunched To 37.00'												
36.0	Prepunched To 37.00'												
36.5	Prepunched To 37.00'												
37.0	Prepunched To 37.00'												
37.5	89.3	84.6	1.24	1.2	369	Med dense, Sand to silty sand	37-40	40-60				21 - 32	20 - 30
38.0	87.0	82.3	1.27	1.3	372	Med dense, Silty sand to sandy silt	37-40	40-60				21 - 32	20 - 30
38.5	91.1	85.9	1.70	1.7	395	Dense, Silty sand to sandy silt	37-40	60-80				21 - 32	20 - 30
39.0	116.3	109.3	1.59	1.3	399	Dense, Sand to silty sand	40-42	60-80				32 - 43	30 - 40
39.5	120.1	112.6	2.02	1.6	407	Dense, Silty sand to sandy silt	37-40	60-80				32 - 43	30 - 40
40.0	138.1	128.1	2.12	1.6	417	Dense, Silty sand to sandy silt	40-42	60-80				43 - 64	40 - 60
40.5	115.7	108.0	2.01	1.6	444	Dense, Silty sand to sandy silt	37-40	60-80				32 - 43	30 - 40
41.0	91.9	85.6	1.65	1.6	450	Dense, Silty sand to sandy silt	37-40	60-80				21 - 32	20 - 30
41.5	85.6	79.5	1.64	1.5	435	Med dense, Silty sand to sandy silt	37-40	40-60				21 - 32	20 - 30
42.0	137.7	127.6	1.87	1.4	477	Dense, Sand to silty sand	40-42	60-80				32 - 43	30 - 40
42.5	117.3	108.4	1.32	1.2	467	Med dense, Sand to silty sand	40-42	40-60				22 - 32	20 - 30
43.0	103.0	94.9	1.32	1.2	473	Med dense, Sand to silty sand	40-42	40-60				22 - 32	20 - 30
43.5	124.3	114.3	1.73	1.4	529	Dense, Sand to silty sand	40-42	60-80				33 - 43	30 - 40
44.0	102.4	93.9	1.54	1.4	551	Med dense, Sand to silty sand	37-40	40-60				22 - 33	20 - 30
44.5	107.6	98.5	1.61	1.2	595	Med dense, Sand to silty sand	40-42	40-60				22 - 33	20 - 30
45.0	175.8	160.5	2.15	1.3	591	Dense, Sand to silty sand	40-42	60-80				44 - 66	40 - 60
45.5	158.8	144.7	2.02	1.2	642	Dense, Sand to silty sand	40-42	60-80				44 - 66	40 - 60
46.0	126.8	115.2	1.83	1.3	670	Dense, Sand to silty sand	40-42	60-80				33 - 44	30 - 40
46.5	119.2	108.1	1.85	1.4	710	Dense, Sand to silty sand	40-42	60-80				33 - 44	30 - 40
47.0	145.8	131.9	1.28	1.1	779	Med dense, Sand to silty sand	40-42	60-80				33 - 44	30 - 40
47.5	136.4	124.9	1.94	1.4	837	Dense, Sand to silty sand	40-42	60-80				33 - 44	30 - 40
48.0	131.7	118.7	1.84	1.4	915	Dense, Sand to silty sand	40-42	60-80				33 - 44	30 - 40
48.5	122.0	109.7	1.80	1.4	1030	Dense, Sand to silty sand	40-42	60-80				33 - 44	30 - 40
49.0	109.5	98.2	1.86	1.6	1072	Dense, Silty sand to sandy silt	37-40	60-80				22 - 33	20 - 30
49.5	122.1	109.3	1.96	1.4	1157	Dense, Sand to silty sand	40-42	60-80				34 - 45	30 - 40
50.0	160.2	143.1	2.24	1.4	1094	Dense, Sand to silty sand	40-42	60-80				45 - 67	40 - 60

Notes:

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Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.

Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

STRATIGRAPHICS

PROJECT NAME: Stratford Army Engine Plant

PROJECT NUMBER: 99-120-050

R2DATE: 5-20-1999 TIME: 10:55:48.01

SOUNDING NUMBER: op-9915

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
50.5	152.8	136.2	1.72	1.1	1121	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
51.0	155.6	138.4	1.58	1.0	1190	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
51.5	151.4	134.3	1.30	0.9	1263	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
52.0	162.6	144.0	1.62	1.0	1345	Dense, Sand to silty sand	40-42	60-80				34 - 45	30 - 40
52.5	153.1	135.3	1.91	1.2	1553	Dense, Sand to silty sand	40-42	60-80				34 - 45	30 - 40
53.0	135.4	119.4	1.82	1.3	1681	Dense, Sand to silty sand	40-42	60-80				34 - 45	30 - 40
53.5	129.8	114.3	1.56	1.2	1699	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
54.0	130.2	114.3	1.50	1.1	1761	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
54.5	123.7	108.4	1.41	1.1	1868	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
55.0	123.0	107.6	1.35	1.1	1962	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
55.5	126.8	110.7	1.35	1.0	2071	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
56.0	139.9	121.8	1.27	0.8	2175	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
56.5	172.1	149.6	1.52	0.9	2216	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
57.0	167.4	145.3	1.08	0.6	2323	Med dense, Sand to silty sand	40-42	40-60				34 - 45	30 - 40
57.5	161.7	137.4	1.44	0.8	2455	Med dense, Sand to silty sand	42-46	40-60				35 - 46	30 - 40
58.0	127.0	109.8	1.47	1.0	2895	Med dense, Sand to silty sand	40-42	40-60				35 - 46	30 - 40
58.5	97.7	84.3	1.97	1.8	2637	Dense, Silty sand to sandy silt	37-40	60-80				23 - 35	20 - 30
59.0	105.3	90.6	4.56	1.0	3725	Med dense, Sand to silty sand	40-42	40-60				23 - 35	20 - 30

* Indicates lightly overconsolidated soil

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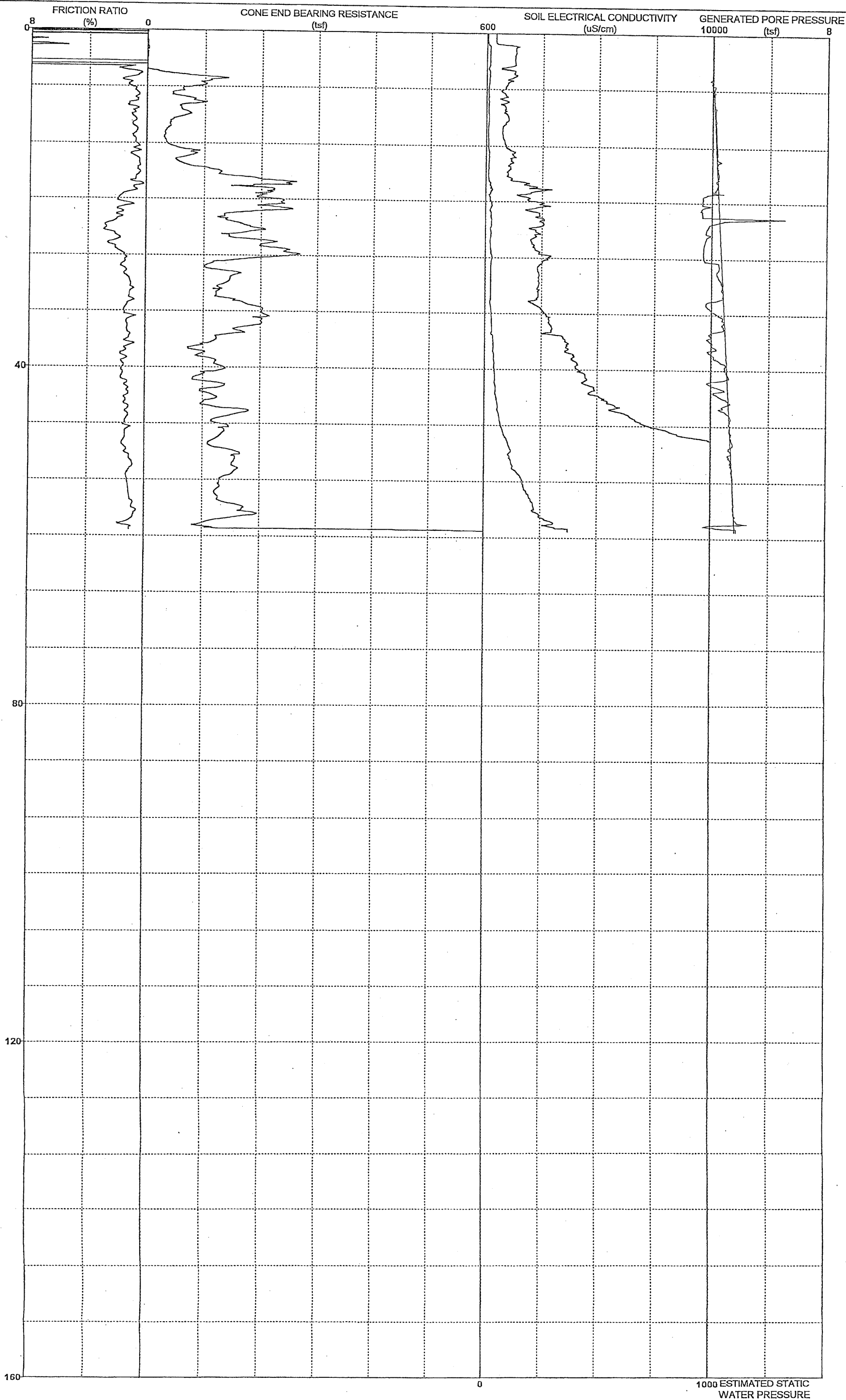
Mixed soils containing both granular and fine grained particles (e.g. clayey sands) may undergo partial drained failure during CPT.
Both undrained and drained parameters can be estimated for these soils.

Structure rate of loading should be considered in choosing which strength parameters to use for design.

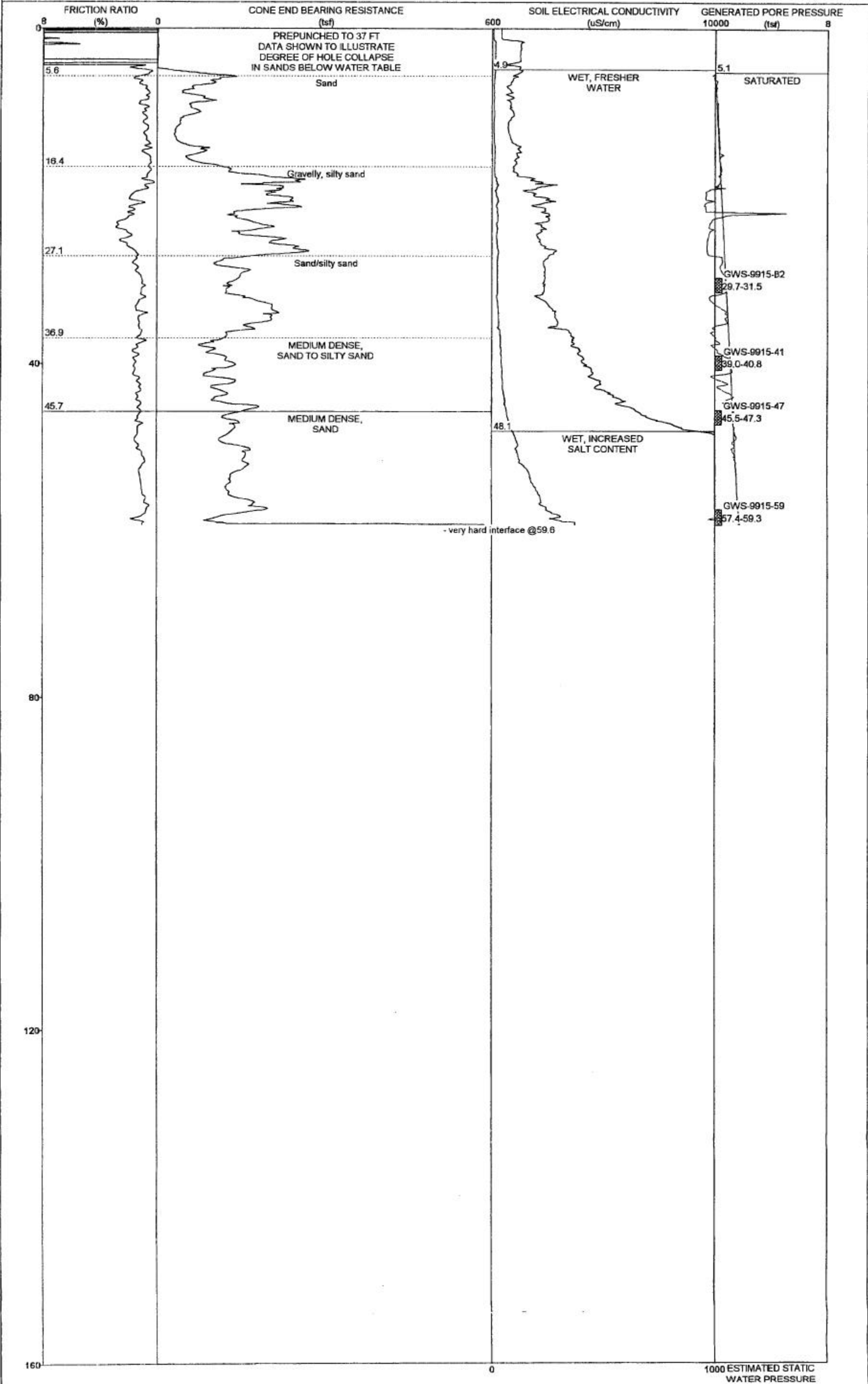
Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

Notes:

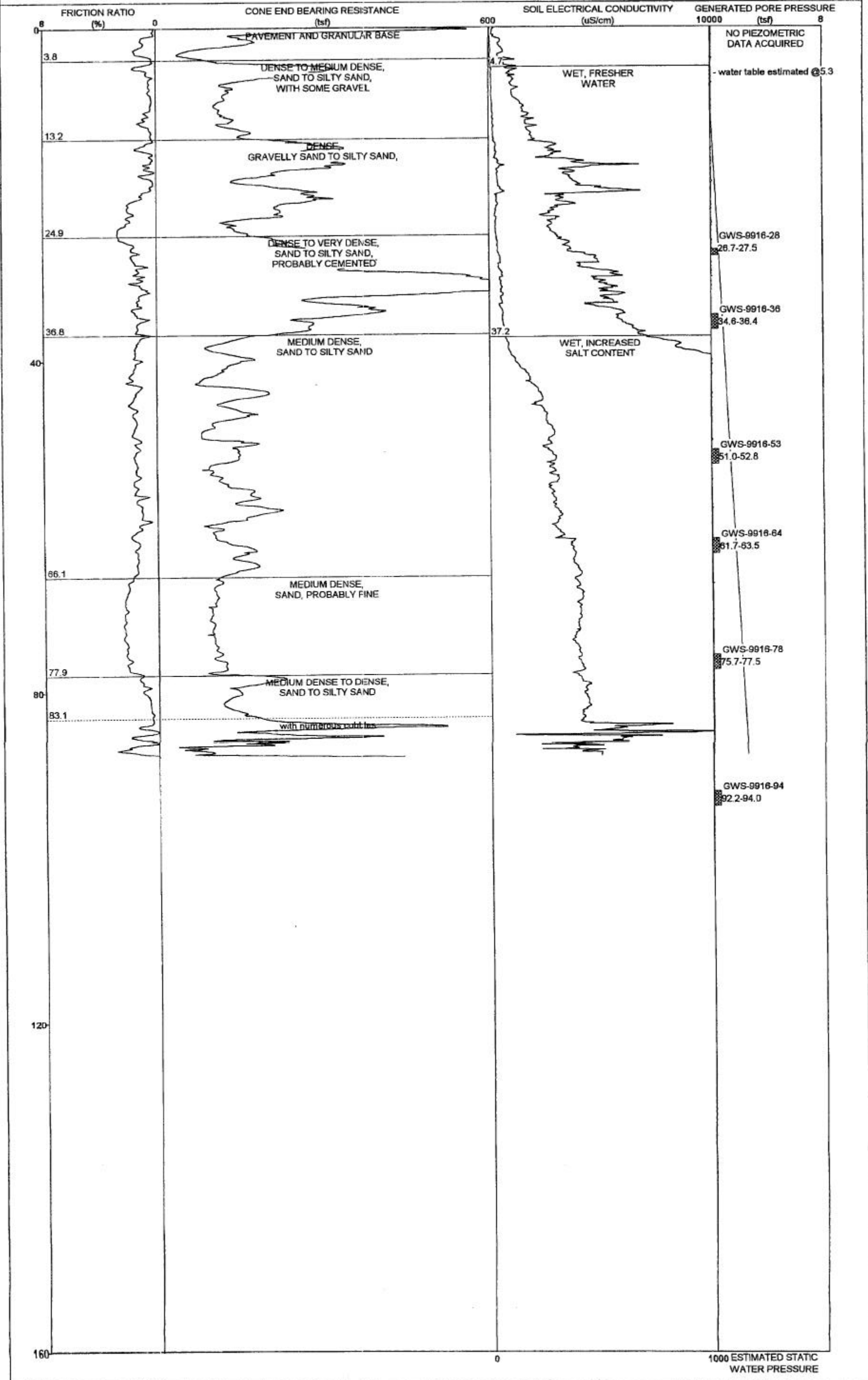
CPTU-EC LOG



CPTU-EC LOG WITH LITHOLOGIC EVALUATION



CPTU-EC LOG WITH LITHOLOGIC EVALUATION



STRATIGRAPHICS

PROJECT NAME:Stratford Army Engine Plant

PROJECT NUMBER:99-120-050

R2DATE: 5-20-1999 TIME:15:45:04.69

SOUNDING NUMBER:CP-9916

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (N1)
50.5	134.9	118.1	2.03	1.3	2376	Dense, Sand to silty sand	40-42	60-80				34-46	30-40
51.0	124.7	124.7	2.22	1.5	2666	Dense, Sand to silty sand	40-42	60-80				34-46	30-40
51.5	143.9	125.5	2.44	1.7	2665	Dense, Silty sand to sandy silt	40-42	60-80				46-69	40-60
52.0	139.0	120.9	2.30	1.6	2678	Dense, Silty sand to sandy silt	40-42	60-80				34-46	30-40
52.5	95.6	83.0	1.75	1.5	2713	Med dense, Silty sand to sandy silt	37-40	40-60				23-35	20-30
53.0	91.5	79.3	1.41	1.5	2706	Med dense, Silty sand to sandy silt	37-40	40-60				23-35	20-30
53.5	95.3	82.4	1.55	1.5	2960	Med dense, Silty sand to sandy silt	37-40	40-60				23-35	20-30
54.0	114.7	99.0	1.32	1.2	3088	Med dense, Sand to silty sand	40-42	40-60				23-35	20-30
54.5	111.3	95.9	1.99	1.7	3097	Dense, Silty sand to sandy silt	37-40	60-80				23-35	20-30
55.0	124.4	107.0	2.27	1.6	2718	Dense, Silty sand to sandy silt	37-40	60-80				35-47	30-40
55.5	168.9	145.0	2.55	1.5	2712	Dense, Sand to silty sand	40-42	60-80				47-70	40-60
56.0	164.6	141.2	2.85	1.6	2911	Dense, Silty sand to sandy silt	40-42	60-80				47-70	40-60
56.5	162.2	155.8	2.12	1.2	2861	Dense, Sand to silty sand	40-42	60-80				35-47	30-40
57.0	140.1	119.6	2.43	1.5	2965	Dense, Sand to silty sand	40-42	60-80				47-70	40-60
57.5	179.7	153.1	2.23	1.1	2963	Dense, Sand to silty sand	40-42	60-80				47-70	40-60
58.0	218.0	185.4	2.39	1.1	3056	Dense, Sand to silty sand	42-46	60-80				47-71	40-60
58.5	178.9	151.9	2.14	1.1	3161	Dense, Sand to silty sand	40-42	60-80				47-71	40-60
59.0	158.0	133.8	1.89	1.1	3080	Med dense, Sand to silty sand	40-42	40-60				35-47	30-40
59.5	109.6	92.7	1.35	1.0	3046	Med dense, Sand to silty sand	40-42	40-60				24-35	20-30
60.0	95.0	80.2	1.29	1.2	2916	Med dense, Sand to silty sand	37-40	40-60				24-36	20-30
60.5	100.7	84.8	1.64	1.5	3038	Med dense, Silty sand to sandy silt	37-40	60-80				24-36	20-30
61.0	101.6	85.4	1.92	1.9	3290	Dense, Silty sand to sandy silt	37-40	60-80				24-36	20-30
61.5	106.1	89.1	1.99	1.7	2966	Dense, Silty sand to sandy silt	37-40	60-80				24-36	20-30
62.0	127.4	106.8	1.80	1.3	3652	Med dense, Sand to silty sand	40-42	60-80				48-72	40-60
62.5	165.1	138.1	2.55	1.5	3681	Dense, Sand to silty sand	40-42	60-80				48-72	40-60
63.0	170.7	142.5	2.48	1.5	3758	Dense, Sand to silty sand	40-42	60-80				48-72	40-60
63.5	141.5	118.0	2.31	1.5	3814	Dense, Sand to silty sand	40-42	60-80				36-48	30-40
64.0	143.0	119.0	2.29	1.4	3733	Dense, Sand to silty sand	40-42	60-80				36-48	30-40
64.5	175.8	146.0	2.70	1.5	3740	Dense, Sand to silty sand	40-42	60-80				48-72	40-60
65.0	170.2	141.2	2.66	1.5	3950	Dense, Sand to silty sand	40-42	60-80				48-72	40-60
65.5	137.1	113.5	2.21	1.5	4023	Dense, Sand to silty sand	40-42	60-80				36-48	30-40
66.0	111.6	92.3	2.19	1.8	4046	Dense, Silty sand to sandy silt	37-40	60-80				36-48	30-40
66.5	111.5	92.0	2.07	1.9	3797	Dense, Silty sand to sandy silt	37-40	60-80				36-48	30-40
67.0	106.4	87.6	2.38	2.1	3978	Dense, Silty sand to sandy silt	36-37	60-80				24-36	20-30
67.5	98.2	80.8	2.10	2.1	3861	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
68.0	102.2	83.9	2.29	2.2	4146	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
68.5	106.6	87.3	2.39	2.3	4132	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
69.0	101.8	83.2	2.48	2.4	4068	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
69.5	101.4	82.8	2.33	2.3	4033	Dense, Silty sand to sandy silt	36-37	60-80				25-37	20-30
70.0	95.0	77.4	2.05	2.2	4030	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
70.5	96.6	78.6	2.31	2.4	4032	Dense, Silty sand to sandy silt	36-37	60-80				25-37	20-30
71.0	92.7	75.4	2.26	2.4	4087	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
71.5	97.9	79.4	2.33	2.4	3962	Dense, Silty sand to sandy silt	36-37	60-80				37-49	30-40
72.0	97.9	79.3	2.43	2.5	3952	Dense, Silty sand to sandy silt	36-37	60-80				25-37	20-30
72.5	95.4	77.2	2.29	2.4	3999	Dense, Silty sand to sandy silt	36-37	60-80				37-50	30-40
73.0	97.3	78.5	2.36	2.4	3828	Dense, Silty sand to sandy silt	36-37	60-80				25-37	20-30
73.5	99.2	80.0	2.43	2.4	3767	Dense, Silty sand to sandy silt	36-37	60-80				37-50	30-40
74.0	99.8	80.3	2.28	2.2	3771	Dense, Silty sand to sandy silt	36-37	60-80				25-37	20-30
74.5	104.9	84.3	2.36	2.3	3915	Dense, Silty sand to sandy silt	36-37	60-80				37-50	30-40
75.0	106.2	85.2	2.43	2.2	3957	Dense, Silty sand to sandy silt	36-37	60-80				37-50	30-40

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PROJECT NAME:Stratford Army Engine Plant

PROJECT NUMBER:99-120-050

R2DATE: 5-20-1999 TIME:15:45:04.69

SOUNDING NUMBER:CP-9916

Depth (ft)	Cone (tsf)	Norm Cone (tsf)	Friction (tsf)	Averaged Friction Ratio (%)	Soil Conductivity (uS/cm)	Evaluated Soil Type	Drained Friction Angle (deg)	Relative Density (%)	Nc	Undrained Shear Strength (ksf)	Undrained Large Strain Shear Strength (ksf)	SPT (N)	NORM SPT (Nf)
75.5	106.5	85.3	2.48	2.3	4071	Dense, Silty sand to sandy silt	36-37	60-80				37 - 50	30 - 40
76.0	104.2	83.3	2.15	1.9	3920	Dense, Silty sand to sandy silt	37-40	60-80				25 - 38	20 - 30
76.5	119.7	95.6	2.69	2.2	4034	Dense, Silty sand to sandy silt	37-40	60-80				38 - 50	30 - 40
77.0	120.5	96.1	2.54	2.1	4118	Dense, Silty sand to sandy silt	37-40	60-80				38 - 50	30 - 40
77.5	95.3	75.9	2.28	1.8	4153	Dense, Silty sand to sandy silt	37-40	60-80				25 - 38	20 - 30
78.0	150.0	151.0	2.86	1.2	3887	Dense, Sand to silty sand	40-42	60-80				50 - 75	40 - 60
78.5	221.7	176.0	2.80	1.3	4065	Dense, Sand to silty sand	40-42	60-80				50 - 76	40 - 60
79.0	178.1	141.2	2.24	1.1	4281	Dense, Sand to silty sand	40-42	60-80				38 - 50	30 - 40
79.5	128.7	101.9	1.13	0.8	4289	Med dense, Sand to silty sand	40-42	60-80				25 - 38	20 - 30
80.0	145.8	115.2	1.58	1.1	4315	Med dense, Sand to silty sand	40-42	60-80				25 - 38	20 - 30
80.5	133.9	105.6	1.08	0.8	4259	Med dense, Sand to silty sand	40-42	60-80				25 - 38	20 - 30
81.0	120.0	94.5	0.81	0.6	4367	Med dense, Sand to silty sand	40-42	60-80				19 - 25	15 - 20
81.5	114.8	90.3	0.73	0.6	4377	Med dense, Sand to silty sand	40-42	60-80				19 - 25	15 - 20
82.0	129.3	101.5	0.85	0.6	4357	Med dense, Sand to silty sand	40-42	60-80				25 - 38	20 - 30
82.5	155.0	121.5	0.62	0.4	4090	Med dense, Sand to silty sand	40-42	60-80				26 - 38	20 - 30
83.0	176.6	136.3	1.04	0.5	4128	Med dense, Sand to silty sand	40-42	60-80				26 - 38	20 - 30
83.5	219.8	171.9	1.32	0.4	4119	Med dense, Sand to silty sand	42-46	60-80				38 - 51	30 - 40
84.0	454.3	354.6	5.60	1.3	4845	V dense, Sand to silty sand	42-46	80-100				+ 128	+ 100
84.5	260.3	202.9	0.80	0.2	5934	Med dense, Sa gravel to gr sand	42-46	60-80				51 - 77	40 - 60
85.0	208.9	162.6	2.12	0.9	9126	Dense, Sand to silty sand	42-46	60-80				51 - 77	40 - 60
85.5	262.6	204.1	4.35	1.6	7633	Dense, Sand to silty sand	40-42	60-80				77 - 127	60 - 99
86.0	157.7	122.4	0.10	0.2	5478	Med dense, Sand to silty sand	42-46	60-80				26 - 39	20 - 30
86.5	59.0	45.7	2.95	2.1	4580	Med dense, Silty sand to sandy silt	27-31	40-60	25	5.98		19 - 26	15 - 20
87.0	80.0	61.9	2.82	2.8	4045	Hard, Sandy silt to sandy clay	42-46	40-60			5.64	28 - 39	20 - 30
87.5	140.4	108.5	0.59	0.0	4946	Med dense, Sand to silty sand	42-46	40-60				19 - 26	15 - 20

Notes:

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Drained and undrained parameters must not be combined as such combination will result in significant overprediction of in situ shear strength.

APPENDIX A
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A1.0 EVALUATION OF GEOTECHNICAL PARAMETERS

CPT data have been correlated with soil type, drained friction angle, undrained shear strength, relative density, and equivalent SPT blowcounts, among others. Correlations have been developed by comparing CPT results to laboratory tests on drilled samples and to other in situ tests, such as vane and pressuremeter. Laboratory CPT testing on large scale samples of known composition and classical bearing capacity and cavity expansion theory have also been used. Site specific information, where available, can be used to fine tune correlations.

A two parameter correlation scheme has proved useful for CPT data interpretation. Geotechnical properties often exhibit well defined trends when plotted against the logarithm of the CPT cone end bearing resistance and friction ratio. For instance, increased grain size increases cone end bearing resistance, while increased plasticity and compressibility increases friction ratio. A generalized chart illustrating these and other trends is presented in Figure A2. An extensive discussion of CPT data evaluation is presented in Douglas and Olsen, 1981.

A1.1 CPT Soil Behavior Types CPT soil behavior type correlations have been developed from geotechnical theory and comparisons of borehole data with CPT data (Douglas and Olsen, 1981). The CPT soil behavior type tabulations are indicative of the response of the soil to the large shear deformations imposed on the soil during penetrometer advance. Soil shear response is not entirely controlled by grain size distribution. However, it has been found that soil types correlated with CPT generally agree with classifications based on soil grain size distribution methods such as the Unified Soil Classification System (USCS). A generalized soil classification chart is presented in Figure A3.

A1.2 CPT Relative Density Relative densities of granular soils are correlated with CPT data on the basis of laboratory CPT on large scale samples of known composition (Schmertmann, 1978, and Villet and Mitchell, 1981). The effect of soil fines content has been empirically accounted for by extrapolating trends in the two parameter correlation model (Douglas and Strutynsky, 1984). A generalized relative density chart is presented in Figure A4.

A1.3 CPT Drained Static Strength Drained friction angles have been correlated with CPT results on the basis of CPT soundings and laboratory tests on drilled samples, and on theoretical analyses of the cone end bearing capacity problem (Schmertmann, 1978, Durgunoglu and Mitchell, 1974, and Villet and Mitchell, 1981). The effect of soil fines content on friction angles has been accounted for by extrapolating trends in the two parameter correlation model, as was done for the relative density correlation. A generalized drained friction angle chart is also presented in Figure A4.

A1.4 CPT Undrained Static Strength The correlation between CPT data and undrained shear strength has been extensively studied (Douglas and others, 1984, Lunne and others, 1976, Sanglerat, 1972, and Schmertmann, 1978). The following bearing capacity equation is used for computing undrained shear strength from CPT data: $q_u = (S_u * N_c) + S_v$ (Eq. A1); where: q_u = ultimate bearing capacity; S_u = undrained shear strength; N_c = a dimensionless bearing capacity factor; and S_v = the estimated total vertical stress. By setting q_u equal to the cone end bearing resistance, q_c , and rearranging the equation, a value of the undrained shear strength can be as: $S_u = (q_c - S_v)/N_c$ (Eq. A2).

The primary difficulty in using this equation has been the selection of N_c applicable to cone penetration in a particular soil. Bearing capacity and cavity expansion theory and other in situ and laboratory test results performed adjacent to CPT soundings have been used to calculate N_c values. These N_c values have ranged from 5 to over 25, but are most often between about 12 and 20. Higher N_c values are typically associated with overconsolidated clays and lower plasticity clays and clayey silts.

A compilation of N_c values as a function of cone bearing resistance and friction ratio is presented in Figure A5. This figure was developed from comparisons of CPT to results of laboratory consolidated-undrained (CU) strength tests. This is important to note as undrained shear strength is not a unique property of a soil - it is test type and stress path dependent.

Many design methodologies are based on a particular strength test on a particular type of sample. These semi-empirical design methods are successfully used by experienced designers. Engineering judgment must be applied in using the results of any type of testing - whether in situ or laboratory - to assure both adequate safety and design economy.

High Strain, Remolded Strength Another measure of the in situ undrained shear strength is provided by the CPT friction sleeve resistance. The friction sleeve interacts with soil that has already undergone bearing capacity failure induced by the tip of the penetrometer. Thus, the friction sleeve resistance is a measure of soil large strain, remolded strength. The ratio between strengths calculated from the cone end bearing and from the friction sleeve is indicative of soil sensitivity.

In moderately to highly overconsolidated, non-sensitive clays, friction sleeve resistances can indicate higher strengths than those calculated using the cone end bearing resistance. This often reflects the dilative (strain hardening) nature of shear failure in overconsolidated soils. Engineering judgment must be applied in deciding which strain level, and thus which strength, is representative for the particular design problem to be solved.

A1.5 Equivalent SPT Blowcount N-Values An equivalent SPT blowcount can be correlated with CPT data by using a mathematical model of the SPT procedure and soil resistances measured during CPT (Douglas and Olsen, 1981). This procedure has been checked by comparison to actual SPT results at various sites throughout the world (Douglas and others, 1981, Douglas and Strutynsky, 1984, and Olsen and Farr, 1986) with generally good results.

The particular SPT equipment used to develop the CPT-SPT correlation chart (Figure A6) consisted of a SPT trip hammer system. This SPT hammer is characterized by reasonably repeatable, measured hammer input energy efficiencies of about 60 to 70% (Douglas and Strutynsky, 1984). This hammer input energy level is similar to that recommended (Seed and others, 1984) as the "standard" Standard Penetration Test input energy. SPT results are both equipment and operator dependent. SPT hammer efficiencies have been measured to range from 35 to over 90% of the theoretical 4200 in-lbs (30 inch height of fall, 140 lbs hammer) SPT input energy. Variable SPT input energy results in variable blowcounts (Douglas and Strutynsky, 1984, Seed and others, 1984). This problem of non-uniform input energy during SPT provides a limitation for quantitative design purposes.

The approach of using the extensive SPT data base, in addition to the CPT data base, by performing CPT and then deriving equivalent SPT blowcount N-values, typically results in higher quality site information. This is because CPT is continuous, has higher resolution, is less expensive, and is much more consistent and repeatable than SPT. The chart that was used for correlating CPT to SPT for this study is presented in Figure A6. After determining the overburden normalized equivalent SPT N'-value, the equivalent SPT blowcount N-value was calculated by dividing the overburden normalized value by the overburden normalization factor CN, as defined in Eq. A3.

The equivalent SPT N-values reflect the higher resolution of the CPT measurements as compared to actual SPT. Performance of actual SPT includes averaging of soil resistance over about a 24 inch interval (18 inch sampler embedment and 2 to 3 sampler diameters ahead of the sampler). Equivalent SPT values correlated with CPT data have a resolution of about six inches. Rather than coarsen the 6 inch resolution equivalent SPT N-value to fit a 24 inch resolution actual SPT N-value, interpreted values are based on point by point CPT data. These high resolution, equivalent SPT values should be more useful for design purposes, especially in interlayered deposits, where thin, weak soil seams cannot be adequately characterized by actual SPT blowcount methods. The high resolution equivalent SPT values and actual SPT measurements should be similar in thick homogeneous strata.

Discrepancies between CPT equivalent SPT N-values and actual, measured SPT N-values are often due to inconsistencies in the performance of actual SPT. Poor fit of CPT equivalent and actual SPT in weak soils with very low blowcounts (0 to 3) can be due to limited accuracy of high capacity CPT loadcells used at the extreme low end of their range, but are more likely caused by extensive borehole disturbance in easily disturbed soil, and set of the SPT sampler under the self-weight of the hammer and drillrods. Discrepancies between equivalent and actual SPT values in very dense or hard soils with high blowcounts, especially in gravelly soils, can be due to both erratic penetrometer or SPT sampler interaction with large soil particles, and basic differences in modes of penetration of the two techniques. Indications of very weak soils, using any method, should strongly encourage additional testing, including undisturbed sampling and sophisticated laboratory testing.

A2.0 OVERBURDEN PRESSURE NORMALIZATION

Overburden normalization of CPT data for correlation purposes is necessary in order to remove the effects of increasing confining pressure with depth on measured results. Cone end bearing resistances can be normalized to an effective vertical overburden pressure of 1 TSF by using the following equations: $qc_1 = qc \cdot CN$ (Eq. A3); and $CN = 1.0 - 0.5 \cdot \log(S_v')$ (Eq. A4); where: qc_1 is the overburden normalized cone end bearing resistance, in TSF; qc is the measured cone resistance, in TSF; CN is the overburden normalization factor; and S_v' is the effective vertical overburden stress in TSF.

Overburden normalization curves are variable (Douglas and Martin, 1980). Most were developed using laboratory CPT and SPT on large scale samples of clean sands, compacted at various relative densities and subjected to various overburden pressures. Application of laboratory results to natural soils may be limited. The CN presented in Equation A4 is similar to that proposed (Seed and others, 1977) for the effect of overburden on SPT blowcounts.

The friction ratio is not normalized based on the assumption that overburden pressure affects friction sleeve and cone end bearing resistance similarly. Since the quantities are divided by each other to compute friction ratio, overburden effects should cancel. Some experience (Olsen and Farr, 1986) indicates that this assumption may oversimplify actual conditions for deep soundings. The friction resistance may be less sensitive to overburden pressure than the cone end bearing resistance. Thus, in soundings deeper than about 100 ft, the friction ratio may gradually decrease with increased penetration, independent of any changes in soil conditions, other than overburden pressure. Due to the variability in overburden normalization curves, no specific correction for overburden pressure on friction ratio has been recommended or used for this study. For this study, effective stresses in Equation A4 were computed using assumed water tables and soil unit weights.

A3.0 TEST DRAINAGE CONDITION

The CPT loading rate is such that drained and undrained conditions exist during penetration of sands and clays, respectively. Partial drainage may occur in mixed soils. Lack of boundary drainage control during any in situ test complicates data analysis, especially in mixed soils, as both frictional and cohesive behavior can be exhibited during testing.

CPTU piezometric data indicate that minor differences in cone end bearing and friction ratio response can correspond with major changes in pore water pressure response during the test (Douglas and others, 1985). The complex volumetric strain field around the penetrometer (Davidson and Boghrat, 1983) precludes reliable geotechnical effective stress analysis of CPTU results in partially drained soil.

Empirical estimates of either drained or undrained parameters can be made in soils composed of mixtures of granular and fine grained particles. These parameters must not be combined - they are to be used alternatively. Combination of the drained and undrained parameters for geotechnical analysis will result in significant overestimation of in situ shear strength.

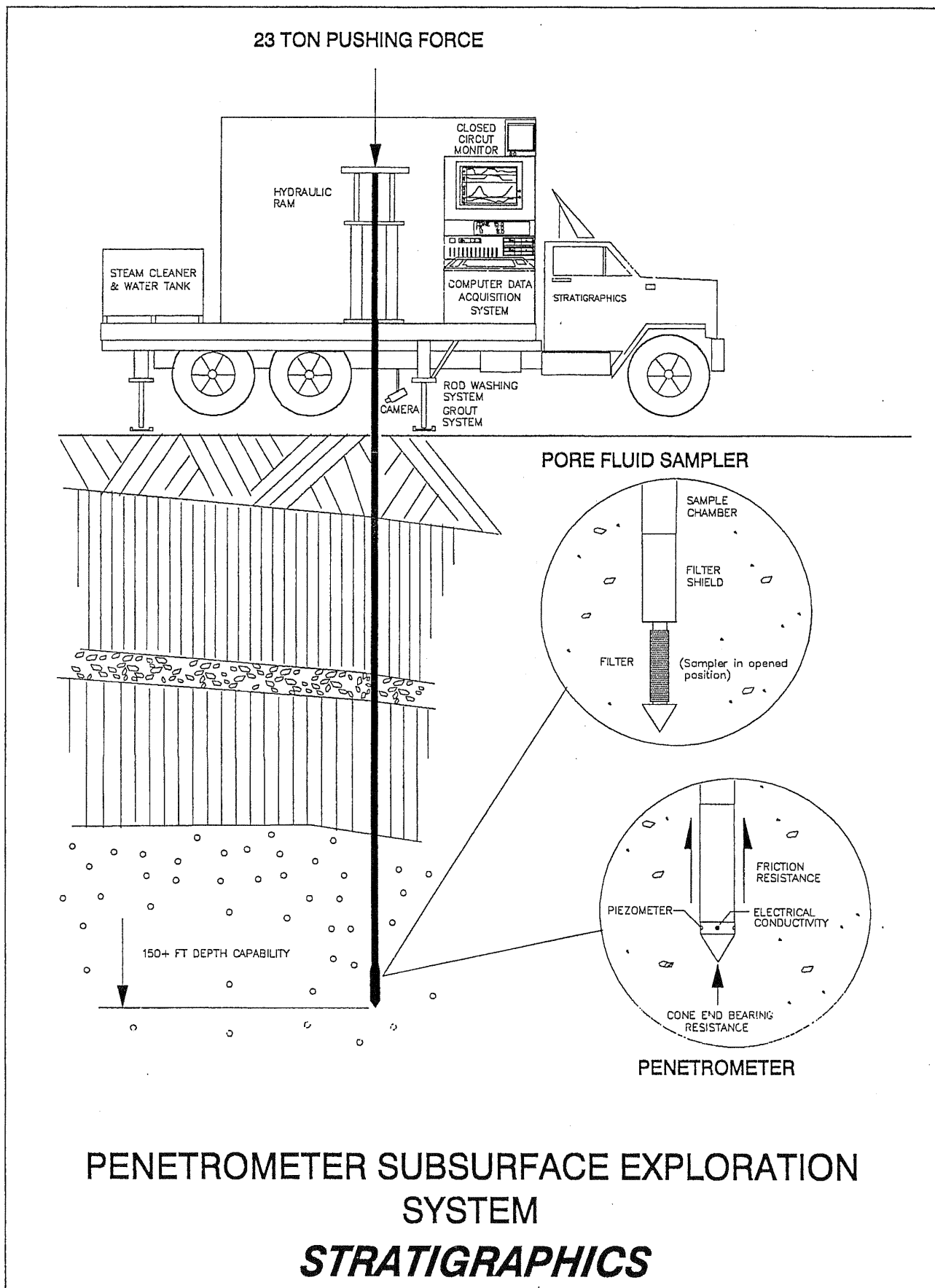
Structure rate of loading will help determine which geotechnical parameters, whether drained or undrained, should be appropriate for design use. Depending on project needs and extent of such soils at a site, geotechnical laboratory testing including CU tests with pore pressure measurements and consolidation tests will also be useful in assigning appropriate design parameters. Field instrumentation during construction using low volume change piezometers may be appropriate for some projects.

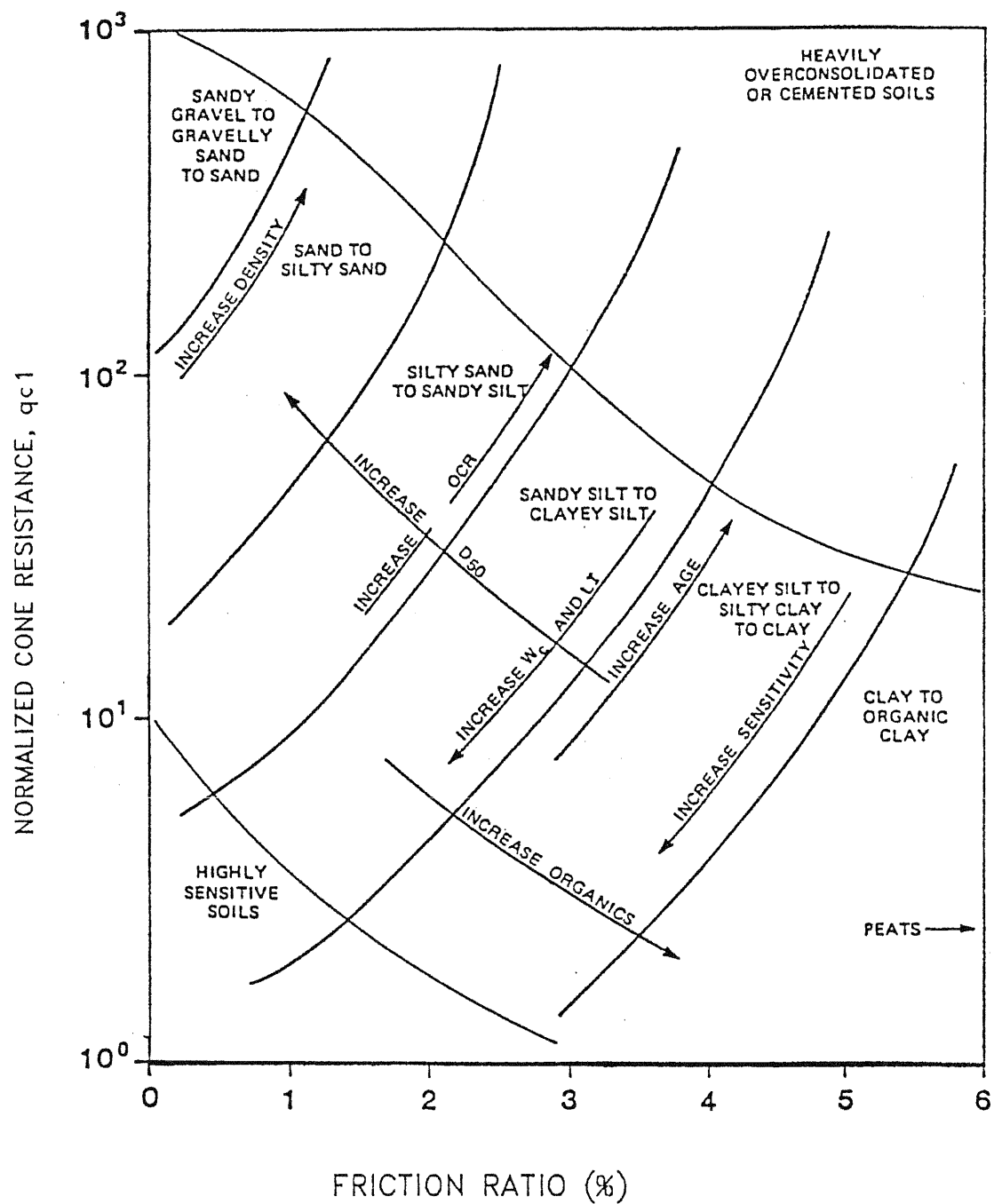
A4.0 RECOMMENDED PRACTICES

The STRATIGRAPHICS parameter evaluation program tracks the CPT data through a series of correlation charts, Figures A2 through A6. Parameters are computer evaluated and tabulated at discrete intervals. Some practices are recommended when reviewing tabulated data and correlated parameters:

- 1) stratigraphic units should be defined on the basis of the continuous sounding logs and project requirements. The tabulations are then used to provide layer properties. Use of the tabulations without the review of the continuous sounding logs can lead to the choice of non-representative parameters, especially in interlayered deposits. It should be noted that taking discontinuous borehole soil samples also often provides a poor representation of subsurface conditions;
- 2) CPT correlations have been developed using empiricism. The empirical data base is world-wide, and includes decades of CPT experience. However, unique local conditions may differ from those in the global data base. Thus, the tabulations should be viewed as indicating trends rather than as the exact equivalent of specific laboratory tests performed under boundary and drainage controlled conditions; and
- 3) while CPT suffers from none of the effects of sample disturbance as found during drilled investigations, boundary conditions are not well defined during any in situ test such as CPT. The derived parameters are not intended to replace appropriate drilling and undisturbed sampling, other in situ and laboratory testing, and use of engineering judgment.

Review of CPT results and project requirements is used to define the need for additional information. Zones delineated by CPT (or, in fact, any other test) providing low factors of safety should be further investigated. Select undisturbed sampling followed by geotechnical triaxial and consolidation testing may be indicated for low strength cohesive or partially drained mixed soil strata. Monitoring wells may be installed or groundwater samples taken in CPT(U) identified high permeability strata during geo-environmental investigations. Laboratory and other test results can then be extrapolated across the site based on CPT(U) defined stratigraphy.





SOIL BEHAVIOR TYPE CLASSIFICATION CHART

After Douglas and Olsen, 1981

STRATIGRAPHICS

Figure 2

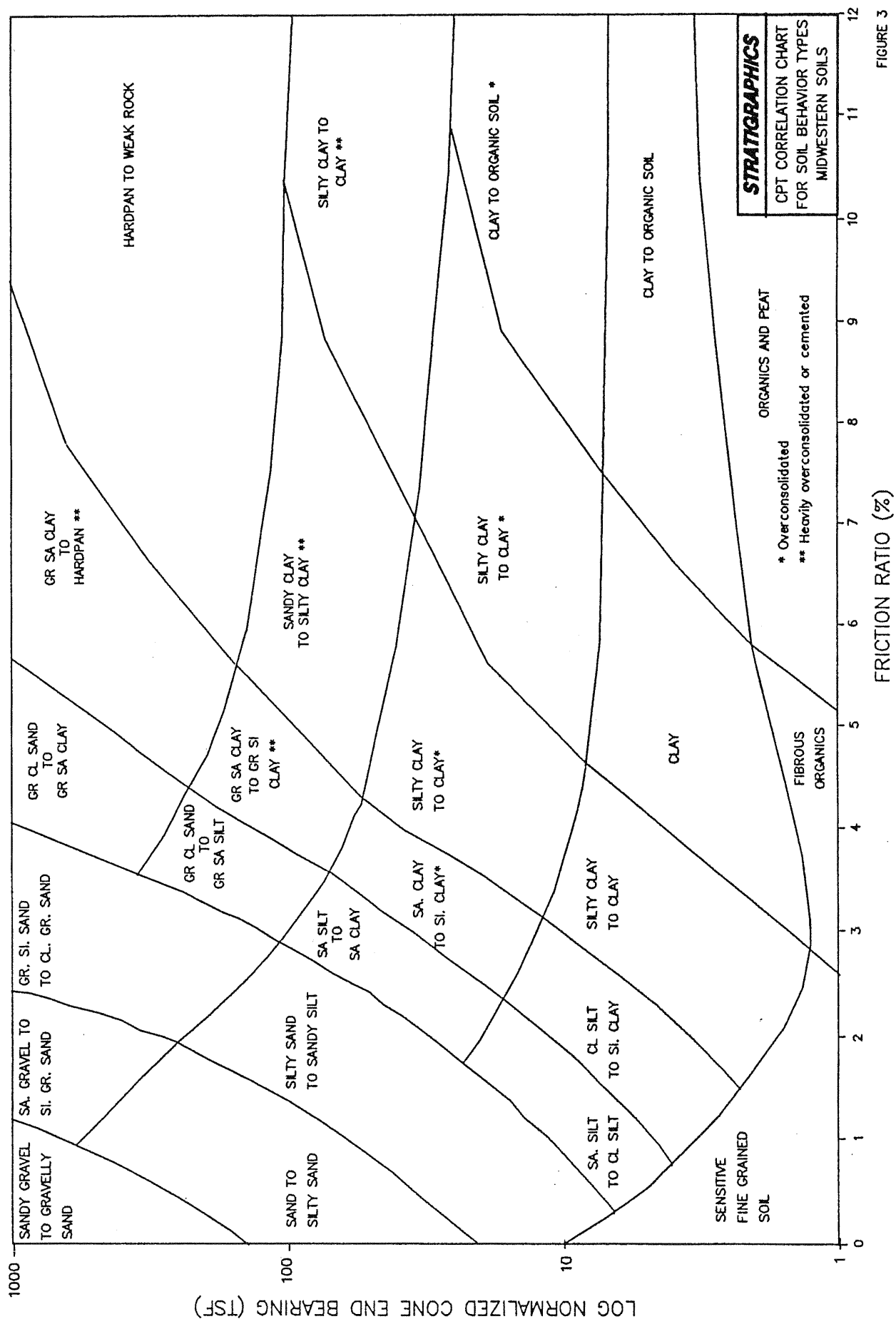
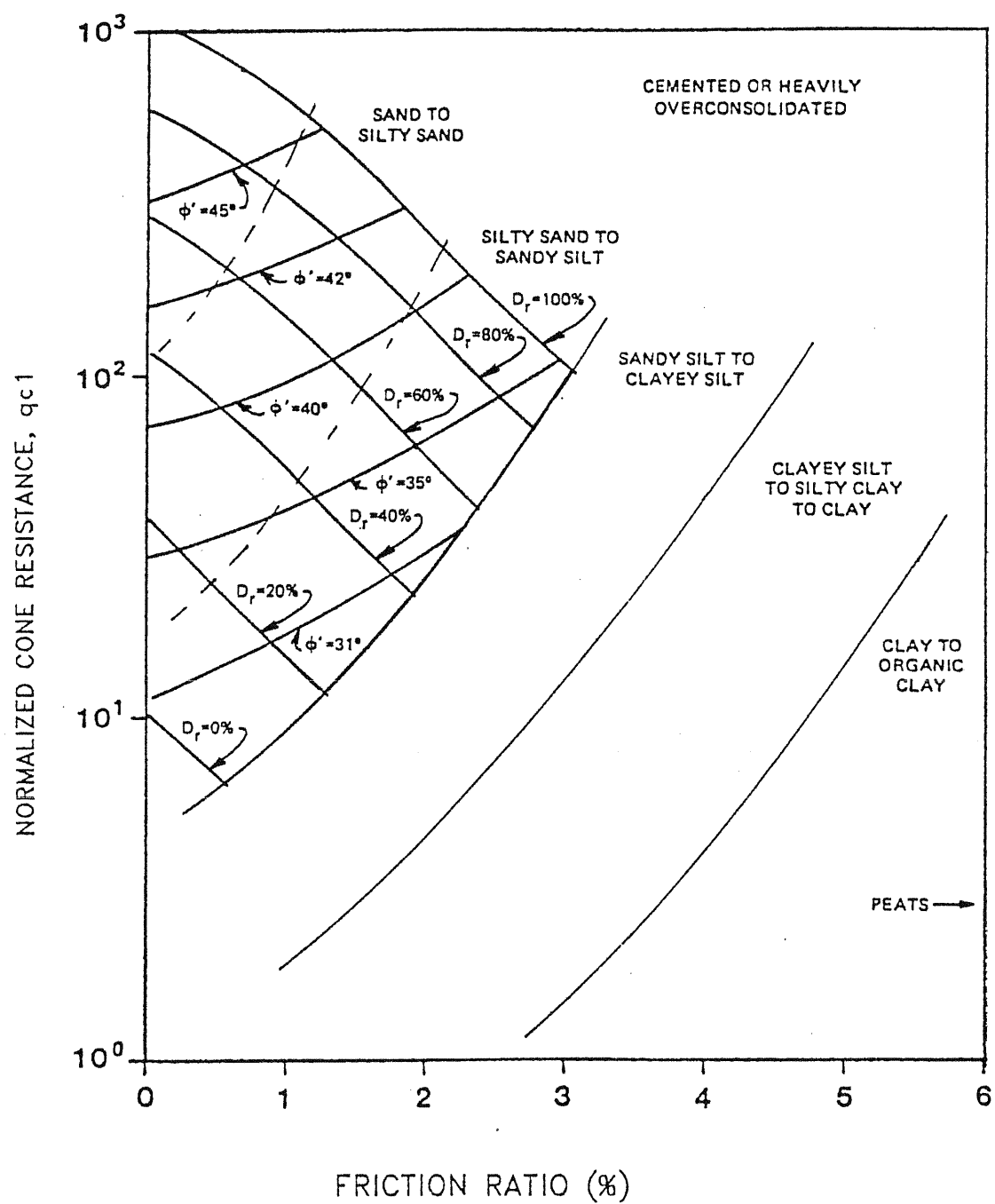


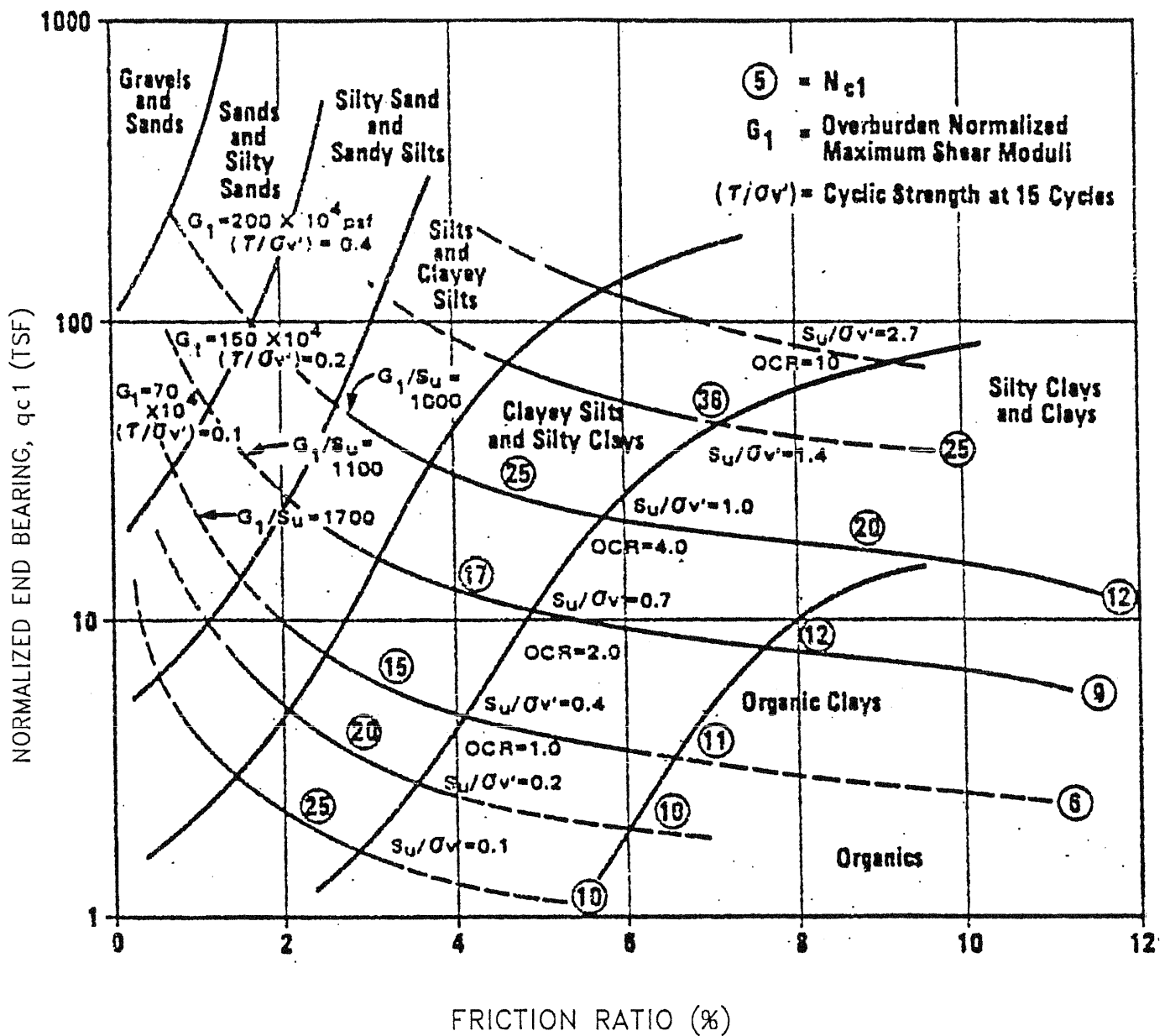
FIGURE 3



EXPANDED SOIL BEHAVIOR TYPE CLASSIFICATION CHART WITH EQUIVALENT OVERBURDEN NORMALIZED FRICTION ANGLE AND RELATIVE DENSITY TRENDS

After Douglas and Strutynsky, 1984

STRATIGRAPHICS



COMPOSITE TRENDS IN UNDRAINED SOIL PROPERTIES

After Douglas, Strutynsky, et. al., 1985

STRATIGRAPHICS

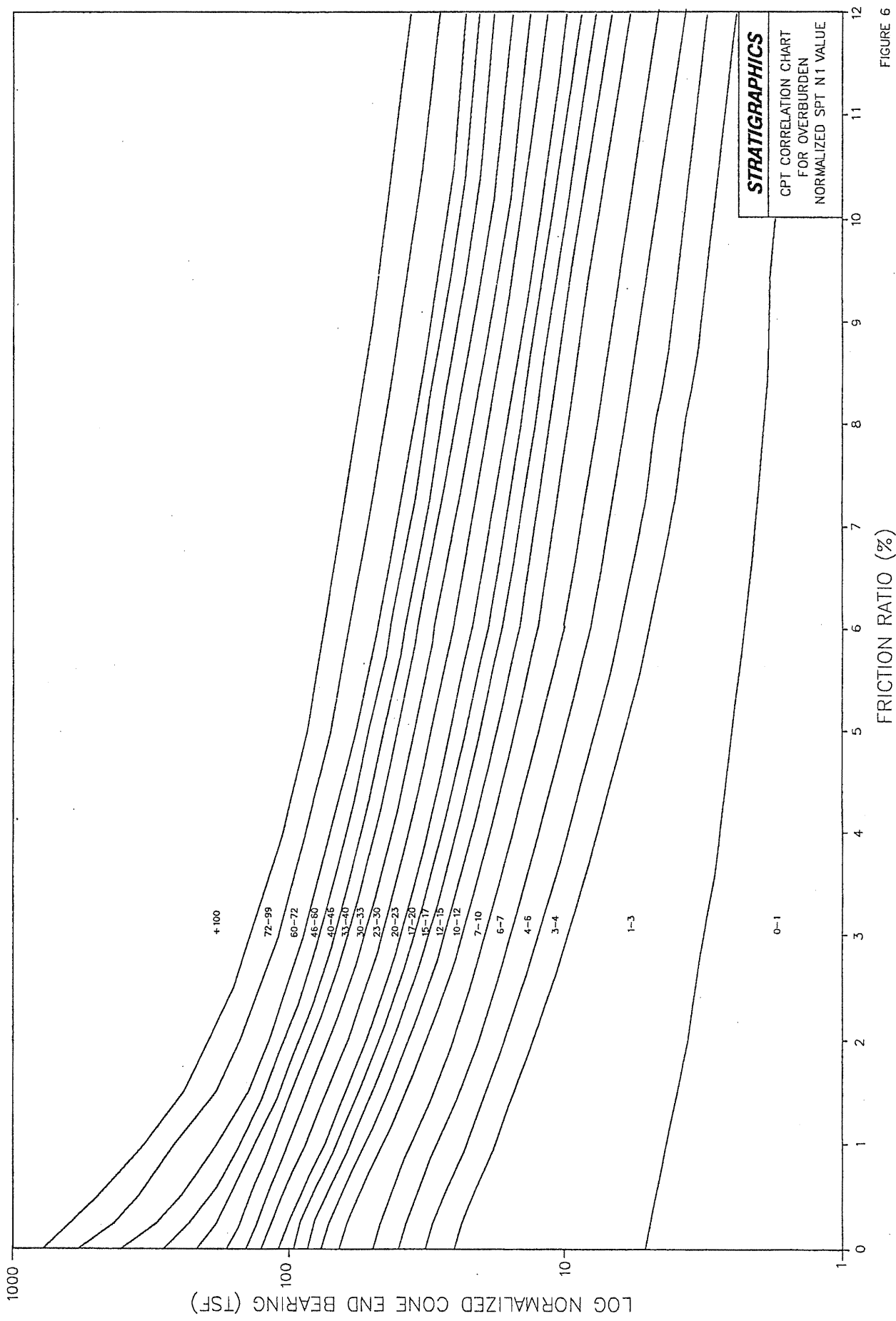
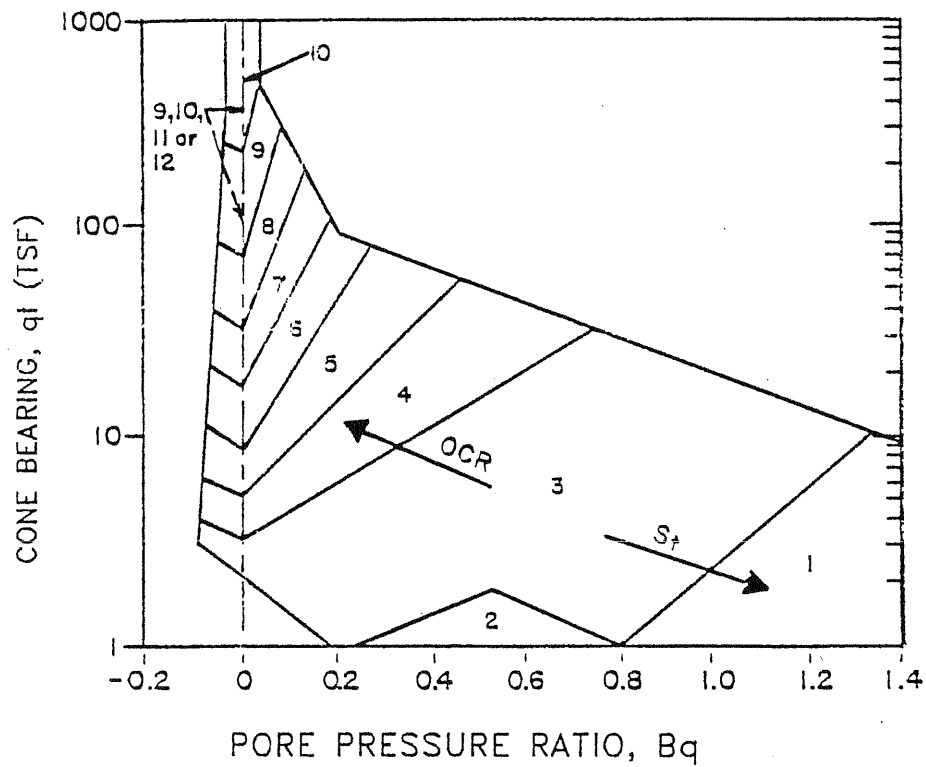


FIGURE 6



ZONE	SOIL BEHAVIOR TYPE
1	ORGANIC MATERIAL
2	CLAY
3	CLAY
4	SILTY CLAY TO CLAY
5	CLAYEY SILT TO SILTY CLAY
6	SANDY SILT TO CLAYEY SILT
7	SILTY SAND TO SANDY SILT
8	SAND TO SILTY SAND
9	SAND
10	GRAVELLY SAND TO SAND
11	VERY STIFF FINE GRAINED (*)
12	SAND TO CLAYEY SAND (*)
(*)	OVERCONSOLIDATED TO CEMENTED

After Robertson & Others, 1986

STRATIGRAPHICS

Figure 7

APPENDIX B

Excerpt from Baligh, M.M. and J. Levadoux, "Pore Pressure Dissipation After Cone Penetration," Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1980.

6.2.4 Evaluation of c_h (probe)

At a given degree of consolidation, the predicted horizontal coefficient of consolidation c_h (probe) is obtained from the expression

$$c_h \text{ (probe)} = R^2 T / t \quad (6.2)$$

where R is the radius of the cone shaft, t is the measured time to reach this degree of consolidation; and T is the time factor. Table 5.1 provides values of T for different probe types at various degrees of consolidation.

An analytical method {equivalent to the graphical method described in Section 6.2.3} to check the validity of the prediction method consists of determining c_h at different dissipation stages, i.e., different u . Large differences between c_h at various degrees of consolidation indicate an inadequate initial distribution of excess pore pressure or significant coupling, or creep behavior.

The estimated values of c_h (probe) at 50% dissipation can be used in foundation problems involving horizontal water flow due to unloading or reloading of clays above the maximum past pressure. For problems involving vertical water flow in the overconsolidated range, the vertical coefficient of consolidation, c_v (probe), can be estimated from the expression:

$$c_v \text{ (probe)} = (k_v/k_h) c_h \text{ (probe)} \quad (6.3)$$

where k_v and k_h are the vertical and horizontal coefficients of permeability, respectively. Reliable estimates of the in situ anisotropy of clays as expressed by the ratio k_h/k_v is difficult to determine in the laboratory because of the effects of sample size, sample disturbance, ... etc. and is the subject of controversy (Rowe, 1972; Casagrande and Poulos, 1969). In situ tests to determine k_h/k_v are almost nonexistent. Table 6.2 provides rough estimates of k_h/k_v for different clays.

6.2.5 Prediction of k_h (probe)

Approximate estimates of the horizontal coefficient of permeability, k_h (probe), can be obtained from the expression:

$$k_h \text{ (probe)} = (g_w / 2.3 s_{v0}) * RR(\text{probe}) * c_h \text{ (probe)} \quad (6.4)$$

where s_{v0} is the initial vertical effective stress (kg/cm^2); g_w is the unit weight of water ($=10^{-3} \text{ kg}/\text{cm}^3$); and, $RR(\text{probe})$ is the recompression ratio during early stages of consolidation around the probe (50% dissipation, say).

Results in both the upper and lower Boston Blue Clays indicate that:

$$\text{the average } RR(\text{probe}) = 10^{-2} \quad (6.5)$$

$$\text{and generally } 0.5 * 10^{-2} < RR(\text{probe}) < 2 * 10^{-2} \quad (6.6)$$

6.2.6 Prediction of $c_v(\text{NC})$

For foundation clays consolidated in the normally consolidated range, estimates of the coefficients of consolidation can be obtained from c_h (probe) by means of the expressions:

$$c_h(\text{NC}) = (\text{RR}(\text{probe})/\text{CR}) * c_h(\text{probe}) \quad (6.7)$$

for horizontal water flow, and

$$c_v(\text{NC}) = (\text{RR}(\text{probe})/\text{CR}) * (k_v/k_h) * c_h(\text{probe}) \quad (6.8)$$

for vertical water flow.

The compression ratio CR is the average slope of the strain vs. log effective stress plot in the appropriate effective stress range expected during consolidation of the foundation clay. Values of CR should be obtained from good quality samples carefully tested in the laboratory. Table 6.2 provides rough estimates of CR based on empirical correlation with index properties of various clays.

Table 6.2 Empirical Correlation and Typical Properties of Clays

1. Compression Ratio CR (from Ladd, 1973)

$\text{CR} = C_c / (1 + e_o) = \text{slope of the strain vs. log stress curve}$

e_o = initial void ratio

C_c = virgin compression index = slope of e vs. log stress

w_L = liquid limit

w_N = natural water content

$C_c = 0.009 (w_L\% - 10\%)$ Terzaghi and Peck (1967)

$C_c = 0.54 (e_o - 0.35)$ Nishida (1958)

$C_c = 0.01 \text{ to } 0.15 (w_N\%)$ MPMR (1958)

$C_c = 0.6 (e_o - 1)$ for $e_o < 6$

$C_c = 0.6 (e_o - 1)$ for $e_o < 6$

$C_c = 0.85 (e_o - 2)$ for $6 < e_o < 14$ Kapp, (1966)

2. Anisotropic Permeability of Clays (from Ladd, 1976)

Nature of Clay	k_h/k_v
1. No evidence of layering	1.2 +/- 0.2
2. Slight layering, e.g., sedimentary clays with occasional silt dustings to random lenses	2 to 5
3. Varved clays in northeastern U.S.	10 +/- 5

APPENDIX C

Brine plume mapping using cone penetrometer and geophysical methods

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ABSTRACT: Naturally occurring brines (concentrated solutions of chloride salt and water) are pumped from deep wells and are transmitted by pipeline to industrial processing facilities. An accidental brine release affected shallow groundwater quality at a site in Michigan. The chloride brine plume was delineated using non-intrusive surface geophysical, minimally intrusive cone penetrometer, and downhole geophysical methods. Surface electromagnetic surveys were used to map the areal extent of the plume. Direct push cone penetrometer soundings with soil electrical conductivity measurements were used to profile stratigraphy and detect electrical conductivity anomalies associated with brine intrusions. Anomalous zones were sampled, using penetrometer groundwater sampling, for direct chemical analysis. Geophysical monitoring stations were then installed in boreholes to allow periodic monitoring of the plume's response to remediation.

1 INTRODUCTION

Soil electrical conductivity (EC) measurements are uniquely useful during geo-environmental exploration of brine contaminated groundwater. Both surface and downhole methods can be used to monitor increases in EC resulting from the migration and diffusion of brine through groundwater. Surface methods have the utility of rapidly covering large areas to identify where brine intrusions might have occurred. Downhole methods are very useful in pinpointing the vertical extent of contamination as well as in estimation of brine concentrations. Downhole measurements can be obtained in cased boreholes, or through use of cone penetrometer soundings which include EC measurements.

Soil electrical conductivity is controlled by the conductance of the soil particles and the conductance of the fluid occupying the soil pore spaces. The ratio between pore fluid and combined soil-pore fluid electrical conductivity is termed the formation factor (Archie, 1942). Clay particles can be electrically conductive due to adsorbed water and ionic electrical charges on the clay platelets, so clay electrical conductance depends on mineralogy, porosity and pore fluid characteristics. Sand grains are typically nonconductive, so sand conductance depends primarily on pore fluid characteristics and

porosity. The addition of brine to groundwater greatly increases soil electrical conductance.

A brine groundwater contamination exploration method was developed based on several technologies. First, surface electromagnetic (EM) surveys can be used to rapidly map affected areas, and give reasonably good indications of the degree of impact of brine releases on groundwater, to depths as great as 60 m (200 ft). However, surface EM methods lack the ability to adequately quantify brine concentrations, cannot provide detailed definition of the depth intervals affected by brine intrusions, and cannot easily differentiate between natural electrically conductive clayey soils and zones of elevated chlorides.

Downhole geophysical induction logging can be performed to great depths in suitably cased boreholes. Data quality is high, allows vertical delineation of affected intervals, and allows estimation of brine concentrations. However, drilling and casing boreholes, and the disposal of large volumes of exploration derived wastes from drilling activities, are expensive and time consuming. Further, borehole stability can be a significant problem where thick sequences of saturated sands are encountered.

A high capacity, truck mounted, geo-environmental cone penetrometer system,

including downhole EC logging and penetrometer groundwater sampling, can be used to vertically profile subsurface conditions and obtain direct groundwater samples. The penetrometer exploration method is fast and accurate, and provides high resolution lithologic and electrical conductivity data. The direct push penetrometer exploration method requires no borehole, so no soil cuttings are generated, and borehole stability is of no concern.

2 GEOPHYSICAL METHODS

The electromagnetic (EM) geophysical method determines electrical properties of earth materials by inducing electromagnetic currents in the ground and measuring the secondary magnetic field produced by these currents. An alternating current is generated in a wire loop or coil above the ground's surface. Both the primary magnetic field (produced by the transmitter coil in the instrument) and the secondary field (produced by currents in the earth) induce a corresponding alternating current in the receiver coil of the instrument. The coils are kept at a fixed distance and orientation relative to the ground to simplify data analysis.

After compensating for the primary field, both the magnitude and relative phase (in-phase and quadrature) of the secondary field are measured.

The quadrature-phase component, using simplifying assumptions of homogeneous and isotropic conditions, is converted to a value of apparent soil electrical conductivity (EC). This value represents an estimate of the local average soil EC. The depth of measurement is dependent on the instrument's coil spacing, orientation, and operating frequency, and the actual subsurface EC variations. Averaging limits discrimination of thin, high concentration brine intrusions from broader, more diffuse plumes. Multiple profiles using differing coil spacing can be performed to bracket approximate depths of brine affected groundwater. Data quality may be degraded by cultural interference as caused by utility lines, steel fences or other large metallic objects.

Surface electromagnetic measurements were taken by Geosphere with Geonics EM38, EM31 and EM34 systems, using coil separations of 1 m (EM38), 3.7 m (EM31), and 10 m or 20 m (EM34). The nominal explored depth is proportional to coil spacing. For the shallow, near field EM38 instrument, the explored depth is about 1.5 m; for the intermediate EM31 it is about 6 m; and for the deep, far field EM34 instrument, the explored depth is about 15 m with a 10 m coil spacing, 30 m with a

20 m spacing, and 60 m with a 40 m spacing.

Instruments were carried manually, and measurements were digitally logged during profiling at the chloride brine release site. Shallow and intermediate readings in the source area were typically taken at 15 m (50 ft) intervals along lines 15 m apart using the EM38 and EM31 instruments. EM34 (deep far field) coverage was conducted on lines spaced 60 m (200 ft) apart in the western portion of the plume, where the chloride brine had migrated to much greater depths.

Permanent geophysical monitoring stations were installed using hollow stem auger drilling techniques. A water filled, PVC casing was grouted into place to provide an access conduit for lowering downhole geophysical logging tools. Screened intervals are not necessary (or even desired) as there is no need for direct communication with the surrounding groundwater/soil system. In induction logging, the field is induced in the surrounding soil through the casing itself. Electrically conductive (i.e., steel) casing materials cannot be used.

Periodic downhole logging of the geophysical monitoring stations was performed at the chloride brine release site using a Geonics EM39 induction system. The induction technique has been used for decades by the petroleum industry for formation characterization in oil and gas wells. The Geosphere application of these methods to shallow groundwater exploration includes increased resolution due to slim, compact logging tools (3.6x130 cm or 1.4x50 in) and slower logging rates. This permits the detection and resolution of formation conductivity changes across vertical intervals of less than 0.3 m (1 ft). The EM39 induction logger is capable of measuring with an accuracy of 2 mS(milli-Siemen)/m in the range of 0 to 1000 mS/m.

The geophysical monitoring station provides significant advantages over traditional screened monitoring wells at brine affected sites. A continuous EC profile is obtained from the surface to the bottom of the cased hole, where traditional monitoring wells only provide data across the screened interval. This is significant as brine plumes typically sink with time, retreating from screened intervals, which can lead to misleading results. The geophysical monitoring station avoids the costs of installing monitoring well clusters, and eliminates data gaps which occur where monitoring wells are screened in different portions of the aquifer. Monitor well purging and analytical testing are also eliminated, leading to additional cost savings. Like surface EM and penetrometer CPTU-EC methods, the borehole EM39 system measures the

bulk (soil and pore fluid) EC. With suitable assumptions of formation factors, a value of groundwater conductivity, and thus approximate concentration of a predominant ion such as chloride, can be estimated from downhole EC measurements.

3 CONE PENETROMETER SYSTEM

The STRATIGRAPHICS cone penetrometer system is used during geo-environmental and geotechnical exploration programs in difficult soil conditions. The heavy (240 kN and 300 kN, or 24 and 30 ton) truck mounted rigs are fully self-contained, and include data acquisition systems, dry and wet work areas, water tanks, steam cleaners, decontamination and grouting systems, separate rodstrings for sounding and sampling, optional dynamic rod driving, and heavy duty downhole equipment for use in glaciated terrains (Figure 1).

Cone penetrometer (direct push) systems require no borehole to advance probes and samplers and result in very little exploration derived waste. Downhole equipment is decontaminated during retrieval using an automatic rodwasher, and the open hole is pressure grouted. Most exploration activities are performed inside an enclosed portion of the rig, providing all-weather capability and a low visual profile during operations. Truck mounted

penetrometer systems can be very productive, with as much as 400 m (1300 ft) of stratigraphic logging per day, with depth capacity exceeding 60 m (200 ft). As many as 18 groundwater, or up to 30 soil or soil gas samples, can be acquired in a day.

Soil resistance to penetration acting on the tip and soil friction on the sides of the penetrometer are separately measured during CPT. These measurements are accurate and repeatable, and have been used for the evaluation of stratigraphy and geotechnical parameters for decades. Performance of geo-environmental CPT is specified by ASTM Standards D5778, D6067 and Guide D6001.

The CPT tip resistance increases exponentially with soil grain size. Tip resistance in a sand aquifer is typically one to two orders of magnitude greater than in a clay aquitard. For example, the CPT tip resistance varies from about 10 to 40 MPa (100 to 400 tons per square ft (tsf)) in a dense sand aquifer, while tip resistance in a stiff clay aquitard ranges from about 0.5 to 1.5 MPa (5 to 15 tsf). The friction ratio (proportion of friction to tip resistance) is also used as an indicator of soil type. The friction ratio allows discrimination between loose sands and hard silts and clays, where tip resistances can be similar. The friction ratio ranges from about 1% in a sand to greater than about 3% in a clay. Silts have intermediate friction ratios.

High resolution, continuous soil profiling (sounding) for geo-environmental exploration is most often performed by STRATIGRAPHICS using the Piezometric Cone Penetration Test with soil Electrical Conductivity measurements (CPTU-EC). The cone tip and friction resistance measurements (CPTU-EC) are evaluated for soil types and geotechnical parameters (Douglas and Olsen, 1981). The piezometric measurement (CPTU-EC) allows evaluation of soil saturation, hydraulic conductivity, potentiometric surfaces, and soil types (Saines et al, 1989, Robertson et al, 1986). The soil Electrical Conductivity measurement (CPTU-EC) provides information on soil moisture in vadose zone soils and indications of groundwater quality in saturated soils. The EC measurement has proved very useful in exploration for inorganic (metal, brines and landfill leachate) contamination and somewhat useful in non-aqueous phase liquid (NAPL) exploration.

Soil EC is measured by STRATIGRAPHICS using a rugged two electrode array mounted on the tip of the penetrometer (Strutynsky, et al, 1991). A 3 kHz AC voltage is applied to the array to control polarization and contact resistance effects. EC is computed based on in-phase currents induced across the array and a reference resistor. The EC

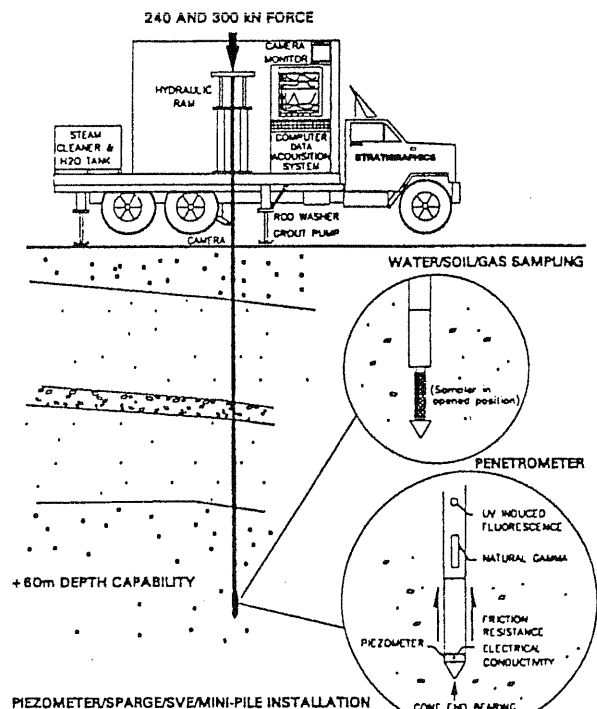


Figure 1 Stratigraphics penetrometer exploration system

measurement has a resolution of about 1-3 cm.

The STRATIGRAPHICS Penetrometer Sampler can be deployed in groundwater, soil or soil gas sampling modes by using interchangeable components. The groundwater sampler is a heavy wall, shielded wellpoint sampler. The shield prevents cross contamination and clogging of the sampler screen. The shield is retracted to allow groundwater to flow through a 0.5 m (20 inch) long screen into the barrel. Sample can be decanted from the barrel or can be pumped to the surface using an inertial pump. A small diameter pressure transducer can be lowered into the sampler to log the rate of groundwater inflow. Inflow results can be analyzed to estimate soil hydraulic conductivity.

Penetrometer exploration can be used at sites where predominant soil grain sizes are less than about $\frac{1}{2}$ the diameter of the downhole tools, i.e. less than about 3-4 cm (medium to coarse gravel). A small fraction of larger particles can be tolerated, as long as the coarser particles are within a matrix of much finer soil grains, as is common in many glacial till units. Deep penetration can be limited by excess friction on the rod string where soft, squeezing clays are encountered. Rod friction can also be a problem where thick sequences of hardpan silts or very fine, saturated sands are encountered. Thick sequences of very soft soils, such as peat layers, can limit deep penetration as little lateral support is provided to the slender penetrometer rod string in the weak layer, while attempting to push through dense soils at depth. Extremely dense (SPT blowcounts greater than about 100) can be difficult to penetrate. Dynamic rod driving may help in this situation.

4 SITE CHARACTERIZATION

Chloride brine production (40% chloride salt solution) from well fields in Michigan is transported to a processing facility through pipelines buried at shallow depths. At one site, galvanic corrosion at a welded pipeline joint led to a release of about 40,000 gallons of chloride brine. The initial response removed over 100,000 gallons of surface water and brine, but a portion of the released brine permeated into a thick sand aquifer, and some seeped into a nearby creek. Chloride brine migration was primarily controlled by differences in density between the heavy brine and lighter fresh groundwater, and by the generally westward regional groundwater flow.

Soils in the release area consist of about 180 m (600 ft) of sands, silts and clays, which overlie

shale bedrock. These soils represent a sequence of lakebed and shoreline deposits in a paleo environment when water levels in the Great Lakes were much higher than at present. The sand aquifer is probably the result of dune and beach activity. The aquifer ranges in thickness from about 6 m (20 ft) near the brine release point, to over 40 m (130 ft) at a distance 600 m (2000 ft) west of the release point. An aquitard consisting of interlayered clays, silts, and silty sands underlies the sand aquifer.

The suspected release point and surrounding area were initially profiled to depths of about 6 m using near field EM38 and intermediate EM31 surface geophysical instruments. Within a week, the shallow plume was mapped to an area of about 35 acres. Purge and monitoring wells were installed to control the migration of the chloride brine through the aquifer. Additional geophysical profiling, and installation of more purge and monitoring wells, was performed in several phases over a period of years, as it became apparent that westward plume migration was continuing.

Due to the dense chloride brine sinking to the top of the westward dipping aquitard layer and the westward regional flow of groundwater, a far field EM34 surface geophysical instrument had to be used for profiling the central portion of the plume. At a distance of about 600 m (2000 ft) west of the release point, the plume had migrated deeper than the layer resolving capabilities of the EM34 instrument. Further plume characterization was performed with the CPTU-EC cone penetrometer method.

A series of 33 CPTU-EC soundings, to depths as great as 50 m (165 ft) and totaling 1061 m (3481 ft) of data, and 27 penetrometer groundwater samples, were completed to characterize the deep, westward extension of the plume. A thin zone of groundwater with slightly elevated chloride concentrations was eventually found to extend along the top of the confining aquitard unit to as far as 1200 m (4000 ft) west of the initial release area.

The indirect CPTU-EC data indicated that the sand aquifer was relatively homogeneous, with very few, thin, apparently discontinuous clay or silt seams or layers. In contrast, the aquitard was found to be much more heterogeneous than the aquifer, with interlayers of clays, silts, and silty sands. EC data (both CPTU-EC and geophysical) indicated groundwater unaffected by the chloride brine is low in electrical conductivity, which provides a very clear contrast to groundwater with elevated chloride content. Piezometric testing during CPTU-EC indicated aquifer hydraulic conductivities to range from about $1\text{E-}3$ to over $1\text{E-}2$ cm/sec. Aquitard

hydraulic conductivities ranged from about $1\text{E-}4$ to less than $1\text{E-}8$ cm/sec, depending on whether the testing was in silty sand or silty clay layers. The indirect penetrometer evaluations of stratigraphy, hydraulic conductivity and EC agreed very well with indirect geophysical, direct sampled borehole, and pump test data collected at the spill site.

The CPTU-EC data were evaluated to delineate the extent of the sand aquifer and clay aquitard units. Depth intervals within aquifer soils, where peak EC measurements were encountered, were selected for direct penetrometer groundwater sampling. Sample targeting using the continuous CPTU-EC sounding logs significantly reduced the number of samples and analytical testing required to adequately characterize groundwater chloride concentrations. Near the plume source, where CPTU-EC measurements were greater than about 400 mS/m (4mS/cm), direct penetrometer groundwater sampling was typically not performed, as high chloride concentrations were not in question for these conditions.

A comparison between CPTU-EC penetrometer data and a drilled, sampled and geophysically logged monitoring station is shown in Figure 2. The two exploration points were within

about a 9 m (30 ft) distance of each other, with the CPTU-EC sounding slightly closer to the plume axis. Very good correspondence is observed between the soil types evaluated from the CPTU-EC measurements and visual classification of obtained samples. The continuous CPTU-EC logs provide data across unsampled intervals in the discontinuously sampled borehole, and greatly increased resolution of thin layering.

Comparison between the CPTU-EC and downhole EM39 data also shows very good correspondence. The CPTU-EC data provide higher resolution (2-3 cm versus 15-30 cm for EM39) of layering. In many instances, this can be advantageous (for example, during other projects when trying to detect thin LNAPL or DNAPL layers). A disadvantage to the high CPTU-EC resolution is that in gravelly soils where particle size is comparable to EC sensor resolution, a significant amount of noise can be exhibited in the EC data.

The CPTU-EC piezometric data in the example sounding log indicate that the bottom of the aquifer unit is slightly less permeable than at shallow depths. This fact, not apparent from visual descriptions of drilled samples, may explain the somewhat unusual shapes of the EM39 geophysical and CPTU-EC logs, and thus the distribution of chloride concentrations at the bottom of the aquifer.

The peak value of EC (and thus, highest chloride brine concentration) might be expected to fall at the interface between the sand aquifer and the underlying aquitard at 25.5 m, if the aquifer was homogeneous in hydraulic conductivity. Instead, the peak EM39 and CPTU-EC values were measured at a depth of about 21 m. This is the depth at which aquifer hydraulic conductivities slightly decrease, as indicated by CPTU-EC piezometric measurements. EC measurements slowly decrease with depth from this point, and somewhat abruptly decrease at about 23 m. The CPTU-EC piezometric data indicate a thin (<10 cm) silt seam at this depth, not detected during drilled sampling. A very abrupt decrease in EC finally occurs at the aquifer/aquitard interface, at 25.5 m.

A comparison between peak EC data measured in the CPTU-EC soundings and average EC data from surface geophysical surveys is presented in Figure 3. Generally good correspondence can be seen between the two exploration methods. As expected, the CPTU-EC peak data are much higher than the average data from the surface EM surveys. The deep, low concentration chloride affected groundwater in the western portion of the site was only detected using

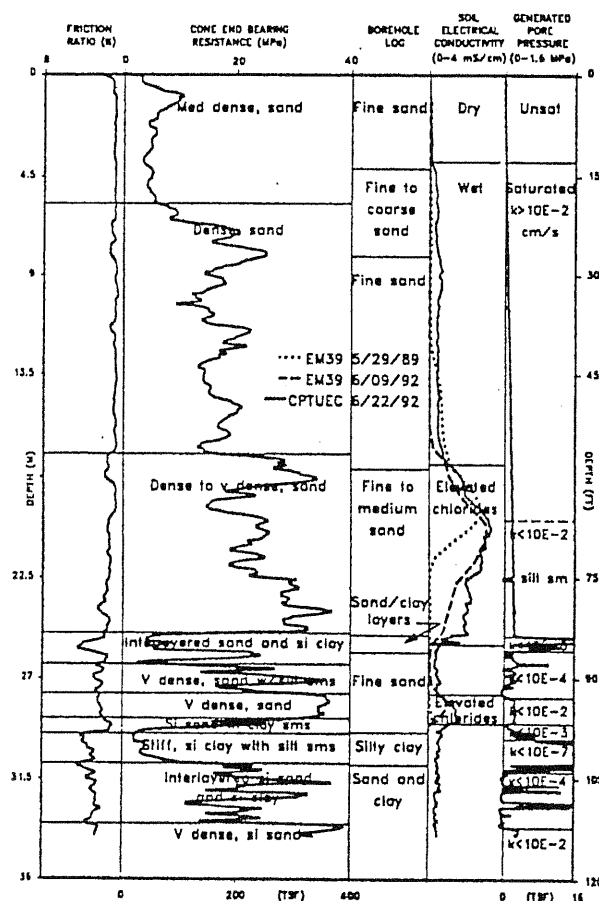


Figure 2 Comparison between CPTU-EC, EM-39 and borehole samples

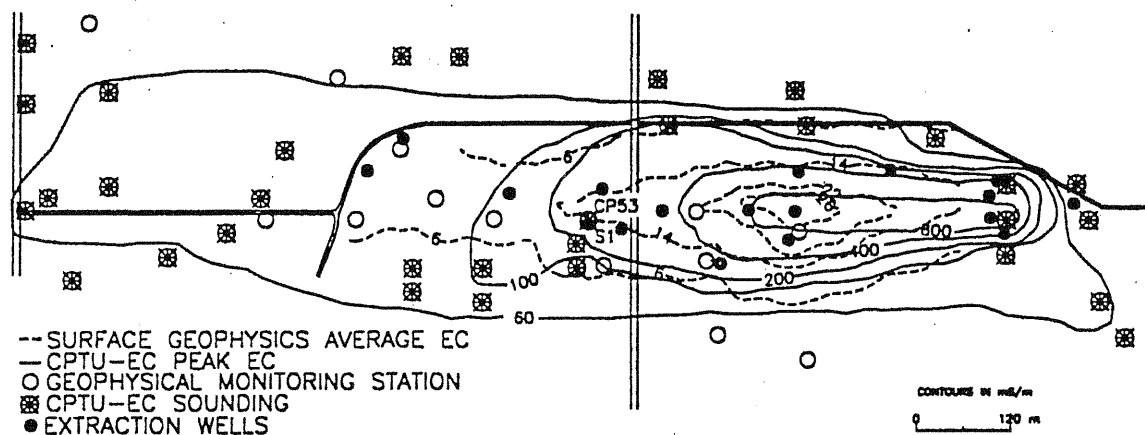


Figure 3 Surface geophysical average EC and CPTU-EC peak EC

CPTU-EC data. The surface EM geophysical survey was of great benefit where plume depths did not exceed about 20-30 m.

Correlations were developed from comparing CPTU-EC and downhole EM39 data to laboratory chloride concentrations obtained from groundwater samples, in order to allow estimation of chloride concentrations. For the CPTU-EC data, and for chloride concentrations less than about 1000 ppm (parts per million), the following relationship was derived: chloride concentration, in ppm = $(0.8 * \text{EC (in uS/cm)}) - 171$. For chloride concentrations up to 50,000 ppm, an alternate relationship was developed: chloride concentration, in ppm = $(1.9 * \text{EC (in uS/cm)}) - 1029$. For a regulatory maximum chloride concentration in groundwater of 250 ppm, the corresponding CPTU-EC value had to be less than about 500 to 550 uS/cm.

It is important to note that these relationships were developed at a site where background groundwater was very low in dissolved minerals, and thus of low electrical conductance. Application of these correlations to other sites is limited - site specific correlations should be performed at each project site.

5 CONCLUSION

An exploration technique for groundwater affected by chloride brine was developed based on surface and downhole geophysical, and direct push cone penetrometer methods. Indirect and direct measurements using these methods were obtained and compared to results of traditional drilled and sampled boreholes and monitoring wells. The new technique provided good technical and economic justification for extensive use at the described site.

Use of the combined geophysical and cone penetrometer exploration technique was then adopted for use at other brine release sites in the area. These programs met with very good success and acceptance by regulatory agencies.

6 REFERENCES

- Archie, G.E., 1942. The Electrical Resistivity Log as an Aid in determining some Reservoir Characteristics. AIME Vol. 146.
- Douglas, B.J., R.S. Olsen, 1981. Soil Classification using the Electric Cone Penetrometer. Cone Penetrometer Testing and Experience, ASCE.
- Robertson, P.K., R. G. Campanella, D. Gillespie, and J. Grieg, 1986, Use of Piezometer Cone Data, In Situ 86, ASCE Specialty Conference, Blacksburg, VA
- Saines, M., A.I. Strutynsky, and G. Lytwynshyn, 1989. Use of Piezometric Cone Penetration Testing in Hydrogeologic Investigations. First USA/USSR Hydrogeology Conference, Moscow, USSR
- Strutynsky, A.I., T. Sainey, 1991. Piezometric Cone Penetration Testing and Penetrometer Groundwater Sampling for Volatile Organic Contaminant Plume Detection. Petroleum Hydrocarbons and Organic Chemicals in Ground water: Prevention, Detection and Restoration. API/NWWA.
- Strutynsky, A.I., R. Sandiford, D. Cavaliere, 1991. Use of Piezometric Cone Penetration Testing with Electrical Conductivity Measurements (CPTU-EC) for Detection of Hydrocarbon Contamination in Granular Soils. Current Practices in Groundwater and Vadose Zone Investigations, ASTM.

GPS General Overview

The Global Positioning System (GPS) may be viewed as continuous radio wave transmissions of extremely accurate (1 second in 30,000 to 300,000 years) timing signals from a constellation of earth orbiting satellites (space vehicles – SV) to a receiver (station) on earth. The time delay from satellite transmission of the timing signal to its reception, along with the velocity of wave propagation (speed of light through the atmosphere) is used to calculate the distance from a satellite to the receiver ($\text{velocity} \times \text{time} = \text{distance}$). Using spherical trigonometry with the calculated distance to at least 4 different satellites, and knowing the precise orbital path of each satellite, the exact position (x, y, z coordinates or latitude, longitude and elevation) of the receiver can be calculated. Satellite orbital information is transmitted along with timing signals from each satellite.

There are three major sources of error in GPS measurements. First is an inadequate number of satellites - a minimum of 4 satellites is required for accurate x, y, and z positioning. Second is errors in broadcast ephemeris, propagation errors through the ionosphere, and receiver clock biases and noise. The third is the intentional corruption of the civilian clear access/coarse acquisition (CA) GPS signal code performed by the US Department of Defense (DOD) for purposes of national defense. The CA signal code corruption is called *selective availability* (SA).

The military (encrypted) portion of GPS has an autonomous accuracy of at least 18 meters horizontally and 30 meters vertically, adequate for the original purpose of GPS - targeting sites with nuclear weapons. With selective availability (SA) corruption of signals, the civilian (CA) portion of the GPS system has an autonomous accuracy of about 100 meters horizontally, and 160 meters vertically. Inexpensive, consumer receivers that only use CA code have this low level of accuracy.

Intermediate cost GPS receivers are available that incorporate a second antenna which allows additional, non-GPS radio wave differential corrections to GPS, allowing horizontal accuracy of about 10 meters. These additional radio wave corrections are available for free from US Coast Guard surface stations, located along various navigable waterways. Subscription (for fee) services based on commercial satellite or cellular tower transmission of these additional radio wave corrections are also available.

Higher cost differential receivers have been developed for accurate surveying purposes that not only use the encoded GPS timing signal, but also account for phasing of the radio frequency carrier wave. These code and carrier phase differential receivers can provide positional accuracy on the order of centimeters. This requires post-processing of recorded GPS data with corrections from a fixed GPS base station. These types of high accuracy receivers with post processing software are used by STRATIGRAPHICS for surveying.

To correct for selective availability (SA) and atmospheric errors, one uses at least two receivers. One receiver (or BASE station) is fixed in position during the survey period, and the other (REMOTE station) is located at the points requiring surveying. Any apparent (and false) motion of the fixed BASE station, due to SA or atmospheric perturbations, is used in corrections to the REMOTE station data set. GPS software is then used to post-process the data, combining satellite measurements from the two receivers to achieve accurate surveying measurements. The accuracy of these post-processed, differentially corrected GPS positional measurements is about 1 ppm of the baseline length (straight-line distance between BASE and REMOTE stations). With a baseline length of 1 mile, this error can be theoretically as low as 2 millimeters.

PROCEEDINGS OF THE FIRST INTERNATIONAL CONFERENCE ON SITE
CHARACTERIZATION – ISC'98/ ATLANTA/GEORGIA/USA/19-22 APRIL 1998

Geotechnical Site Characterization

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VOLUME 1



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Groundwater drinking supply protection using cone penetrometer methods

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ABSTRACT: The Ohio Environmental Protection Agency (OEPA) has the responsibility to oversee public water systems in Ohio. When chlorinated solvents were detected in the municipal wellfield of Bridgeport, OEPA decided to try cone penetrometer exploration methods to study the problem. Penetrometer stratigraphic profiling, with groundwater, soil and soil gas sampling, and field GC/MS chemical analysis, were used to map the plume impinging on the wellfield, and locate the source of the plume. This method was then applied to studies of contaminant plumes affecting the wellfields of Coal Grove and the Belmont County Sanitary Sewer District #3, among others. This paper will present penetrometer equipment descriptions, exploration program philosophy, and short case histories of these investigations

1 INTRODUCTION

OEPA investigations were undertaken at the Bridgeport, Coal Grove and Belmont County Wellfields, among others, to locate the sources of contaminant plumes affecting the wellfields. Another investigation objective was to gather data to evaluate potential response actions to ensure safer water supplies for the future. The choice of exploration method would be primarily controlled by four factors: 1) cost effective plume delineation - it was important that stratigraphy and groundwater chemistry be rapidly evaluated, to allow subsequent exploration points to be located based on actual site conditions, rather than on an arbitrary grid pattern. This approach would avoid expensive, multi-phased exploration typical of many groundwater studies; 2) very low contaminant concentrations - this would require use of relatively sophisticated field analytical equipment; 3) exploration in urbanized areas - it was important that methods be as unobtrusive as possible; and 4) geological conditions - wellfield aquifers were 15-30 m (50-100 ft) deep, and both gravelly and heaving sands might be encountered.

The consultant (Lawhon and Associates) recommended, and OEPA approved, use of a cone penetrometer exploration system (Strutynsky and Sainey, 1991) along with a field analytical laboratory to achieve the goals of the program. The consultant and OEPA provided senior professionals to immediately evaluate all exploration results and plan

subsequent exploration locations, while the penetrometer company (STRATIGRAPHICS) provided a geotechnical engineer or hydrogeologist to evaluate stratigraphy and recommend sampling procedures. OEPA also provided oversight of the entire program.

Initial groundwater samples would be collected around the most contaminated well or lateral, in the case of the Ranney Well, to determine the direction of plume transport. A series of upgradient exploration lines would then be run transverse to the expected axis of the plume to delineate the plume and identify its source. A number of penetrometer soundings, depending on encountered geological conditions, would be performed to develop stratigraphic cross-sections. Groundwater samples would be collected at multiple depths at each location to determine the vertical distribution of contamination within the aquifer. Field analytical testing would allow a 3-dimensional profile of the plume to be rapidly developed, allowing subsequent exploration locations to be chosen based on the complete analytical database. Penetrometer soil and soil gas sampling could be performed, in addition to groundwater sampling, to confirm potential source areas.

2 CONE PENETROMETER SYSTEM

The STRATIGRAPHICS cone penetrometer system

was designed in 1986 for use during both geo-environmental and geotechnical exploration programs in difficult soil conditions. The heavy (240 kN and 300 kN or 24 and 30 ton) truck mounted rigs are fully self-contained, including data acquisition systems, dry and wet work areas, water tanks, steam cleaners, decontamination and grouting systems, separate rodstrings for sounding and sampling, optional dynamic rod driving, and heavy duty downhole equipment for use in glaciated terrains (Fig 1). Cone penetrometer (direct push) systems require no borehole to advance probes and samplers, and result in little exploration derived waste. Downhole equipment is decontaminated during retrieval using an automatic rodwasher, and open hole is pressure grouted. Most exploration activities are performed inside an enclosed portion of the rig, providing all-weather capability and a low visual presence. Truck mounted penetrometer systems can be very productive, with as much as 400 m (1300 ft) of stratigraphic logging per day, with depth capacity exceeding 60 m (200 ft). As many as 18 groundwater, or up to 30 soil or soil gas samples can be acquired in a day.

High resolution, continuous soil profiling (sounding) for geo-environmental exploration is most often performed by STRATIGRAPHICS using the indirect Piezometric Cone Penetration Test with soil Electrical Conductivity (CPTU-EC, Fig 2). The cone tip and friction sleeve resistance measurements (CPTU-EC) are evaluated for soil types and geotechnical parameters (Douglas and Olsen, 1981). The piezometric measurement (CPTU-EC) allows evaluation of soil saturation, hydraulic conductivity, potentiometric surfaces, and soil types (Saines et al, 1989). The soil Electrical Conductivity measurement (CPTU-EC) provides information on soil moisture in vadose zone soils and indications of groundwater quality in saturated soils. The EC measurement has proved very useful in exploration for inorganic (metal, brines and landfill leachate) contamination (Strutynsky et al, 1998) and somewhat useful in LNAPL or DNAPL exploration (Strutynsky et al, 1991).

The STRATIGRAPHICS Penetrometer Sampler can be deployed in groundwater, soil or soil gas sampling modes by using interchangeable components. The groundwater sampler is a heavy wall, shielded sampler. The shield prevents cross-contamination of the sample and screen clogging. The shield is retracted to allow groundwater to flow through a 0.5 m (20 inch) long screen into the barrel. Sample can be decanted from the barrel or can be pumped to the surface using an

inertial pump. The sampler is typically tripped out after each sample to allow thorough decontamination. The rate of groundwater inflow and equilibrium levels can be recorded using a small

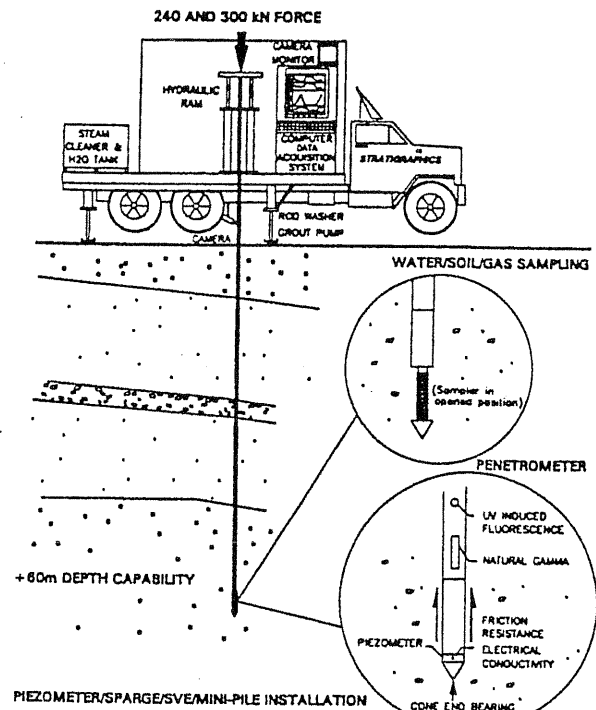


Figure 1 Stratigraphics penetrometer exploration system

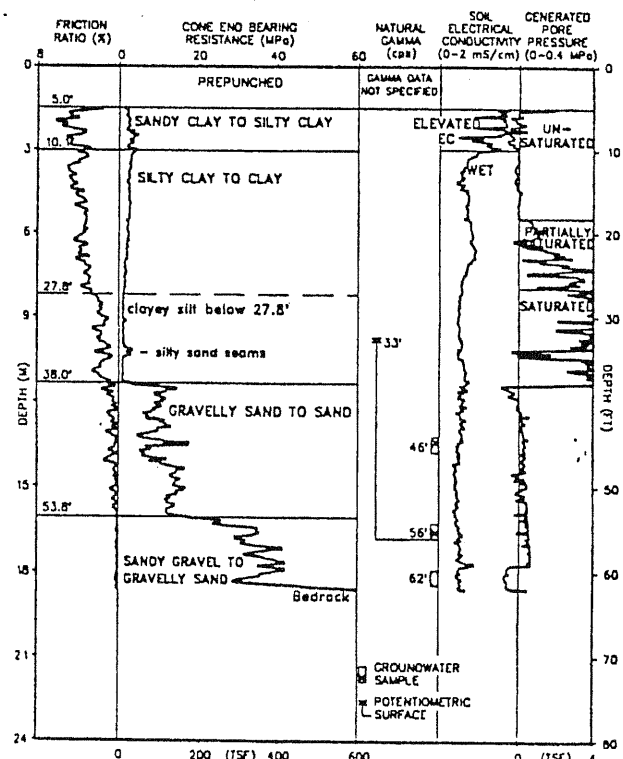


Figure 2 Example stratigraphics CPTU-EC sounding log

diameter pressure transducer. Inflow test results can be analyzed using rising head slug test solutions. The soil sampler consists of a barrel, sealed with a locking piston. The piston is unlocked with a wireline tool to obtain the sample. The sampler is tripped out of the hole for sample extrusion and decontamination. The soil gas sampler is a smaller version of the groundwater sampler. Samples are contained within Tedlar bags, or are routed through portable vapor analyzers.

3 ANALYTICAL TESTING

A Hewlett-Packard Model 5890 Series II Gas Chromatograph (GC) and a Hewlett-Packard Model 5971 Series Mass Spectrometer (MS) were operated by the analytical company (Aqua Tech Environmental Laboratories- ATEL) in a trailer mounted, on-site field laboratory. A Tekmar 2000 purge and trap device, and a Tekmar 2016 auto sampler for automated sample handling, were also used. The samples were analyzed using EPA method 524.2 with a detection limit of 0.5 ug/l. The contaminants of concern (COC's) varied for each site, but were primarily chlorinated solvents, and their breakdown products.

4 BRIDGEPORT WELLFIELD INVESTIGATION

The Bridgeport wellfield is located in southeastern Ohio, along a channel of the Ohio River, and serves 3600 residents. The wellfield is within the unglaciated Appalachian Plateau Ohio River Aquifer. Beginning in 1989, very low levels of TCE and cis-1,2-DCE were detected during routine wellfield monitoring; soon PCE was also detected. After multiple well sampling events confirmed the continual presence of the COC's, OEPA decided to perform an investigation during the summer of 1994.

Ten CPTU-EC soundings for stratigraphy, 26 dissipation tests for hydraulic conductivity and potentiometric surfaces, 52 groundwater (from 57 attempts), and 9 soil gas samples were acquired during the course of a 13 day field program. Soil gas samples were obtained in vadose zone soils around the suspected source after the groundwater sampling program defined the limits of the plume.

Evaluation of the CPTU-EC soundings revealed variability (sand and cinder fills, buried refuse, or clay) at shallow depths. Shallow soils were typically moist to wet. Deeper stratigraphy was more uniform, and typically consisted of sands

and silty sands, with some local gravelly layers. Saturated conditions were typically found below depths of 10-13 m (35-45 ft). The aquifer was characterized as a continuous water table aquifer, with few, apparently discontinuous, aquiclude interlayers. Bedrock was typically encountered within about 21-26 m (70-85 ft) of the surface.

CPTU-EC soundings in the identified source area indicated somewhat different stratigraphy, with finer grained soils predominating, and significant thicknesses of interlayered sands, silts and clays. Bedrock was also found at a shallower depth (18 m or 59 ft), consistent with a steep slope of a buried bedrock valley, as indicated by nearby rock outcrops.

Plume concentration maps were developed at three different depth intervals. In the upper interval, concentrations ranged from less than 0.5 ug/l, to as high as 9,700 ug/l total COC's at the source area. The upper portion of the plume angles away from the wellfield following regional ground water flow. In the middle interval, concentrations ranged from less than 0.5 ug/l, to as high as 1,900 ug/l total COC's near the center of the plume (Fig 3). In the lower interval, concentrations ranged from less than 0.5 ug/l to 105 ug/l total COC's, again with the highest concentrations near the center of the plume. The results indicated that while most of the COC's are drawn towards the wellfield through the middle depth interval, contamination of the wellfield is actually occurring in the lower depth interval.

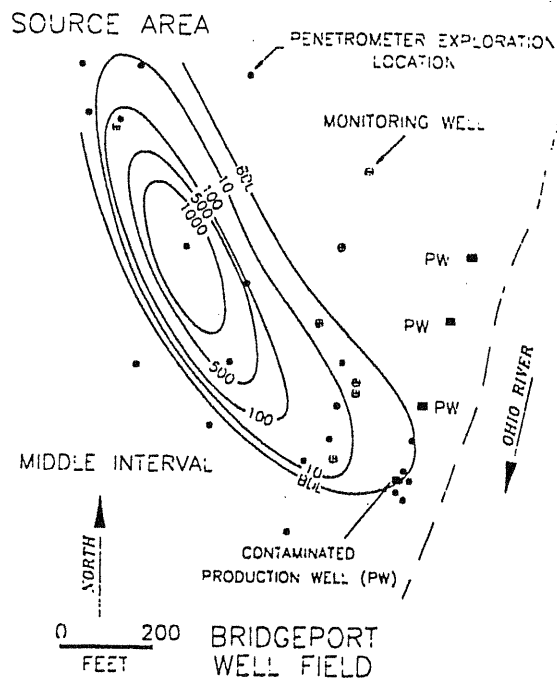


Figure 3

The investigation identified the source of the contamination as a dry cleaner located about 365 m (1200 ft) northwest of the wellfield. OEPA positioned 6 permanent monitoring wells along the plume. These are used as an early warning system, allowing wellfield operators to modify wellfield production to lessen capture of the plume.

5 COAL GROVE INVESTIGATION

The Coal Grove wellfield is located on the banks of the Ohio River in southern Ohio. The four production wells are configured within about 1/2 hectare (1.5 acres), and serve 4700 residents. The production wells are within alluvial deposits associated with the Ohio River and a tributary, Ice Creek. TCE and DCE have been detected in some of the production wells since 1988. One production well (CG-2) was taken out of service in 1989 due to high contaminant levels, and has been intermittently pumped to waste to control contamination of the other production wells. Five monitoring wells had been installed upgradient of the wellfield. TCE and DCE detections increase with distance upgradient.

Upgradient (southeast) of the wellfield is a coal dock. Further upgradient is a closed facility which had operated for numerous years as a truck terminal and as a tanker truck repair and cleaning operation. In 1993, USEPA conducted an emergency removal action at this facility. During the removal action, on-site wastes were found to contain various volatile organic compounds (VOC's), including TCE. OEPA determined that additional groundwater data were required to protect the wellfield from further damage. The penetrometer method successfully used at Bridgeport was chosen for use at Coal Grove. The investigation had to proceed in two parts (1994 and 1995) due to lack of permission to enter the tanker truck cleaning facility.

Fourteen CPTU-EC soundings for stratigraphy, 38 dissipation tests for hydraulic conductivity and potentiometric surfaces, and 61 groundwater samples (from 64 attempts) were acquired during the course of the first 12 day field program. CPTU-EC soundings revealed 5.5-14 m (18-46 ft) of silty clay, underlain by granular soils. The gravelly sand to silty sand aquifer saturated thickness varied from about 1.2 m (4 ft) downgradient to 13 m (43 ft) upgradient of the wellfield. The aquifer ranged from water table to confined, depending on surficial clay thickness. Bedrock was encountered at the base of the aquifer, at depths between 17-26 m (55-87 ft).

The results from field GC/MS analytical testing on obtained samples showed that the VOC plume followed a linear flow path, apparently originating to the southeast of the coal dock facility, on the northern boundary of the closed tanker truck cleaning facility (Fig 4). The plume narrowed with distance upgradient of the wellfield. The ability to rapidly obtain data from the field analytical lab, as well as stratigraphic information from the cone penetrometer, was instrumental in locating this narrow plume. As the investigation proceeded, it became apparent that if sampling had been strictly conducted on the sample grid which was originally surveyed at the site, the narrow, most contaminated portion of the plume would have been entirely missed, with significantly different conclusions as to the source of groundwater contamination.

To complete this investigation and confirm the source of the VOC's, the first program was used to support an administrative search warrant for exploration at the closed tanker truck cleaning facility. With the unexpected accompaniment of local TV news reporters, the warrant was served, the property was entered, and additional exploration was conducted. Four CPTU-EC soundings for stratigraphy, 6 dissipation tests for hydraulic

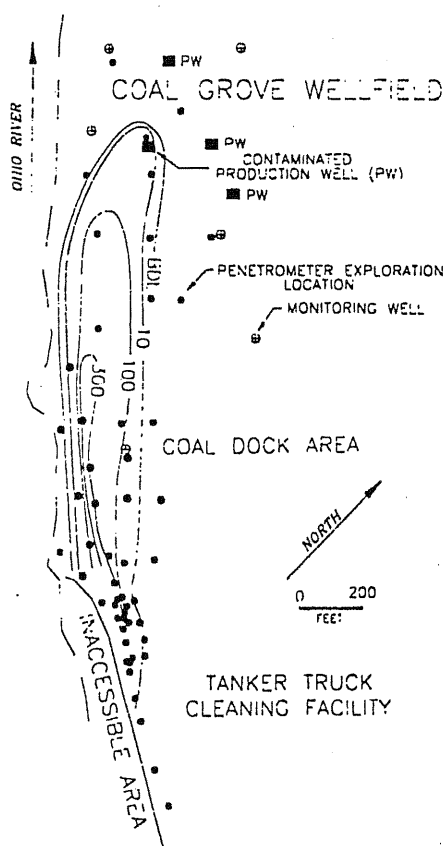


Figure 4 Source area

conductivity and potentiometric surfaces, 37 groundwater samples (from 37 attempts), and 30 soil samples were acquired during the second 10 day field program.

The second program showed that the high concentrations of VOC's continued in a very narrow plume less than 15 m or 50 ft wide to a corner of the closed tanker truck cleaning facility's wastewater treatment plant. This confirmed that the source of the VOC contamination was the tanker truck cleaning facility. Following this sampling effort, OEPA conducted a pump test at the wellfield. The data obtained from the pump test, along with the groundwater analytical data, were used to evaluate future impacts to the wellfield. OEPA concluded that contaminant levels at the wellfield could be expected to remain constant for some time into the future. However, it was also concluded that, without the continued pumping to waste of production well CG2, higher contaminant levels would be expected in other production wells. OEPA is currently evaluating both technical and legal means to address the threat to the Coal Grove wellfield.

6 BELMONT COUNTY SANITARY SEWER DISTRICT 3 RANNEY WELL INVESTIGATION

The Belmont County Sanitary Sewer District 3 serves about 25,000 people in southeastern Ohio with water from a single Ranney Collector Well (BC#3). This well is located near the Ohio River, and consists of six laterals, which are screened near bedrock. The well can supply up to 25 million liters (6.5 million gallons) of water per day. Contaminants cis-1,2-DCE, 1,1,1-TCA, PCE and 1,1-DCA have been detected in the well.

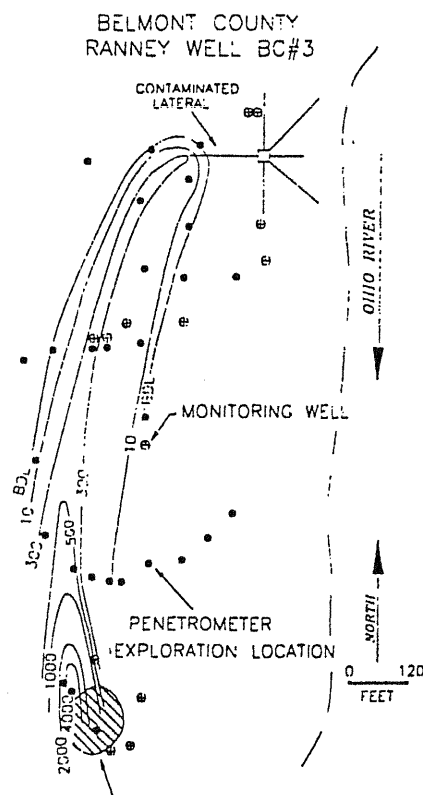
The Ohio Department of Transportation (ODOT) discovered VOC contamination in soil and groundwater during an environmental investigation of a nearby property (Site) during a highway relocation project. The Site is about 365 m (1,200 ft) southwest of BC#3. Various chemicals, including chlorinated solvents and petroleum products, were distributed from a bulk plant located at the Site. ODOT had detected the following VOC's in Site soils and groundwater: VC; PCE; TCE; cis-1,2-DCE; trans-1,2-DCE; 1,1,1-TCA; and BTEX compounds. Initial results indicated that VOC's were migrating in ground water north-northeast from the Site toward BC#3, and towards the southeast with regional groundwater flow.

ODOT notified OEPA of its findings in early 1996. During the fall of 1996, OEPA performed a

field investigation to achieve the following goals: 1) identify all sources of BC#3's contamination; 2) determine the vertical and horizontal extent of contaminant plumes, 3) determine rates of migration; and 4) determine whether the suspect Site was impacting BC#3. The investigation was successful in achieving all these goals.

A major portion of the investigation was conducted using penetrometer exploration methods developed during the Bridgeport and Coal Grove investigations. Six CPTU-EC soundings for stratigraphy, 8 dissipation tests for hydraulic conductivity and potentiometric surfaces, and 55 groundwater samples (from 58 attempts) were acquired during the course of the 10 day field program. The CPTU-EC soundings revealed fine grained soils to depths of about 11-14m (38-47 ft), followed by a confined, gravelly sand aquifer. Groundwater sampling was conducted to determine the lateral and vertical extent of contamination within the aquifer.

The most common groundwater contaminant was found to be cis-1,2-DCE, with concentrations as high as 4,200 ug/l. The following cis-1,2-DCE detections were observed between the Site and BC#3: 300 ug/l about 135 m (440 ft) south of BC#3;



SOURCE SITE
Figure 5

360 ug/l about 40 m (140 ft) south of BC#3; and 90 ug/l about 15 m (50 ft) south of BC#3. Data showed that the plume was being drawn northward from the Site, directly against the regional ground water flow direction, and into the end of the western lateral of BC#3 (Fig 5). This unusual, reversed flow path reflects the very large capture zone of the high capacity Ranney Well. The investigation showed that VOC's are migrating from the Site to BC#3 via ground water flow paths primarily in the upper portion of the aquifer. It also showed that the Site is apparently solely responsible for the BC#3's cis-1,2-DCE contamination.

ODOT and OEPA agreed that excavation and off-site disposal of contaminated soils and debris prior to highway construction would best remediate the Site. Source removal, as opposed to treatment, was selected both to complete the highway project on time and to eliminate further contamination of the aquifer. The source removal took place during the summer of 1997. About 20,000 tons of contaminated, non-hazardous, solid waste and about 1,550 tons of hazardous waste were excavated and removed from the Site.

7 CONCLUSIONS

Thirty four CPTU-EC soundings, totaling 685 m (2250 ft) of data; 205 groundwater, 9 soil gas, and 30 soil samples (3755 m or 12210 ft of sampler deployment) were obtained during the 45 days of field exploration for these 3 projects. Penetrometer costs (less mobilizations) totaled about \$133,000. A total of 244 samples, plus numerous QA/QC samples, were analyzed by the field laboratory using GC/MS. The cost for the field analytical laboratory totaled about \$45,000 (less mobilizations). While this cost is comparable to costs for off-site analyses with a 24 hr turnaround, having analytical results within 15-30 minutes of sample acquisition allowed optimal placement of subsequent exploration locations. Accurate targeting of exploration locations was the key factor in significantly decreasing overall cost and duration of these investigation programs.

OEPA rapidly and cost-effectively investigated contaminated municipal wellfield groundwater supplies by the use of high capacity penetrometer stratigraphic profiling, rapid penetrometer groundwater, soil and soil gas sampling, a sophisticated field analytical laboratory, and immediate evaluation of acquired data by senior professionals. Complex contaminant plumes, which

often followed unusual groundwater flow paths, were characterized within days rather than months or years. Plumes were delineated, sources identified, and realistic models developed for planning long term monitoring and site remediation activities. OEPA has used these projects during in-house training sessions as examples of innovative and effective groundwater exploration.

REFERENCES

Douglas, B.J., R.S. Olsen, 1981. Soil Classification using Electric Cone Penetrometer. Cone Penetrometer Testing and Experience, ASCE.

Saines, M., A.I. Strutynsky, and G. Lytwynyshyn, 1989. Use of Piezometric Cone Penetration Testing in Hydrogeologic Investigations. Presented at the First USA/USSR Hydrogeology Conference, Moscow, USSR.

Strutynsky, A.I., T.J. Sainey, 1991. Use of Piezometric Cone Penetration Testing and Penetrometer Groundwater Sampling for Volatile Organic Contaminant Plume Detection. Petroleum Hydrocarbons and Organic Chemicals in Groundwater: Prevention, Detection and Restoration. API/NWWA.

Strutynsky, A.I., R. Sandiford, D. Cavaliere, 1991. Use of Piezometric Cone Penetration Testing with Electrical Conductivity Measurements (CPTU-EC) for Detection of Hydrocarbon Contamination in Saturated Granular Soils. Current Practices in Ground Water and Vadose Zone Investigations, ASTM.

Strutynsky, A.I. R. Glaccum, L. Conklin, and B. Baker, 1998. Chloride Mapping using Geophysical and Cone Penetrometer Methods. Proceedings of the First International Conference on Site Characterization, Atlanta, GA

PROCEEDINGS OF THE FIRST INTERNATIONAL CONFERENCE ON SITE
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Geotechnical Site Characterization

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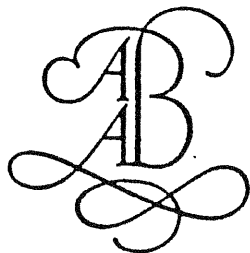
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VOLUME 1



A.A. BALKEMA/ROTTERDAM/BROOKFIELD/1998

Andrew I. Strutynsky, Raymond E. Sandiford, and Dennis Cavaliere

USE OF PIEZOMETRIC CONE PENETRATION TESTING WITH ELECTRICAL CONDUCTIVITY MEASUREMENTS (CPTU-EC) FOR THE DETECTION OF HYDROCARBON CONTAMINATION IN SATURATED GRANULAR SOILS

REFERENCE: Strutynsky, A.I., Sandiford, R.E., and Cavaliere D., "Use of Piezometric Cone Penetration Testing with Electrical Conductivity Measurements (CPTU-EC) for the Detection of Hydrocarbon Contamination in Saturated Granular Soils," Current Practices in Ground Water and Vadose Zone Investigations, ASTM STP 1118, David M. Nielsen, Martin N. Sara, Editors, American Society for Testing Materials, Philadelphia, 1991.

ABSTRACT: Piezometric Cone Penetration Testing with soil Electrical Conductivity measurements (CPTU-EC) was used for the detection of hydrocarbon saturated granular soils at two airport fuel storage areas. Details of the CPTU-EC equipment and site subsurface conditions are provided. Test program phases, including laboratory testing, field insitu testing and CPTU-EC data interpretation are described. Comparisons are made between CPTU-EC and adjacent monitor well data. Limitations to the CPTU-EC method are discussed.

KEY WORDS: Piezometric Cone Penetration Test, Soil Electrical Conductivity, free phase petroleum, hydrocarbon product contamination, airport fuel tank farms, computerized data acquisition, continuous soil profiling.

INTRODUCTION

Programs to remediate ground water accumulations of free phase petroleum hydrocarbon products, consisting primarily of aviation jet fuels, are ongoing at the fuel tank farms at John F. Kennedy (JFKIA), La Guardia, and Newark International Airports, in and around the city of New York. The Port Authority of New York and New Jersey (Port Authority) frequently requires supplemental ground water information in addition to that acquired in monitor wells at the sites.

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STRATIGRAPHICS, a consulting company specializing in penetrometer data acquisition, was retained by Port Authority to evaluate the use of penetrometer soil electrical conductivity measurements (CPTU-EC) to delineate the accumulations of hydrocarbon products in the subsurface at the sites. STRATIGRAPHICS personnel had previously conducted similar studies on using penetrometer conductivity measurements for the detection of hydrocarbon contaminated soils and for the detection of ground water ice crystals in Arctic permafrost soils (Reference 1).

The penetrometer technique provides various advantages during geo-environmental subsurface investigations. These advantages include a relatively non-destructive test procedure; immediate, computerized data reporting and interpretation; continuous profiling; a high degree of exploration personnel safety; and lower exploration costs and higher productivity as compared to borehole techniques.

The experimental program for evaluating the applicability of penetrometer soil electrical conductivity measurements for the detection of free phase petroleum hydrocarbon products in ground water consisted of two phases. The first was a laboratory study using typical site soils, ground water, and jet fuel. A series of 60 tests was performed in order to establish a range of expected field measurements. The initial laboratory study was followed by field studies at the Satellite and Bulk Fuel Farms at JFKIA.

CPTU-EC soundings were performed adjacent to monitor wells for comparisons between penetrometer and monitor well data. CPTU-EC soundings were also performed at intermediate locations for correlation to the areal distribution of hydrocarbon product accumulations. A total of 48 CPTU-EC soundings were performed during the field study (Figures 1 and 2).

SOIL ELECTRICAL CONDUCTIVITY

Soil electrical conductivity is controlled by the conductance of the system of soil particles and fluids occupying the soil pore spaces. Factors affecting soil electrical conductivity, especially for sand aquifers, include:

Mineralogy Siliceous sand grains are essentially non-conductive, so granular soil electrical conductance is dependent on the quantity and conductance of the soil pore fluid. Clay minerals have some electrical conductance due to adsorbed water and ionic charges, thus clay conductance depends on both mineralogy and pore fluid characteristics.

Pore Fluid The electrical conductance of pore fluids plays the major role in granular soil electrical conductivity. Sands saturated with conductive fluids, such as saline water or landfill leachates, have a relatively high conductivity. Sands saturated with petroleum hydrocarbon products typically have low electrical conductivity because most petroleum hydrocarbon products are poor conductors.

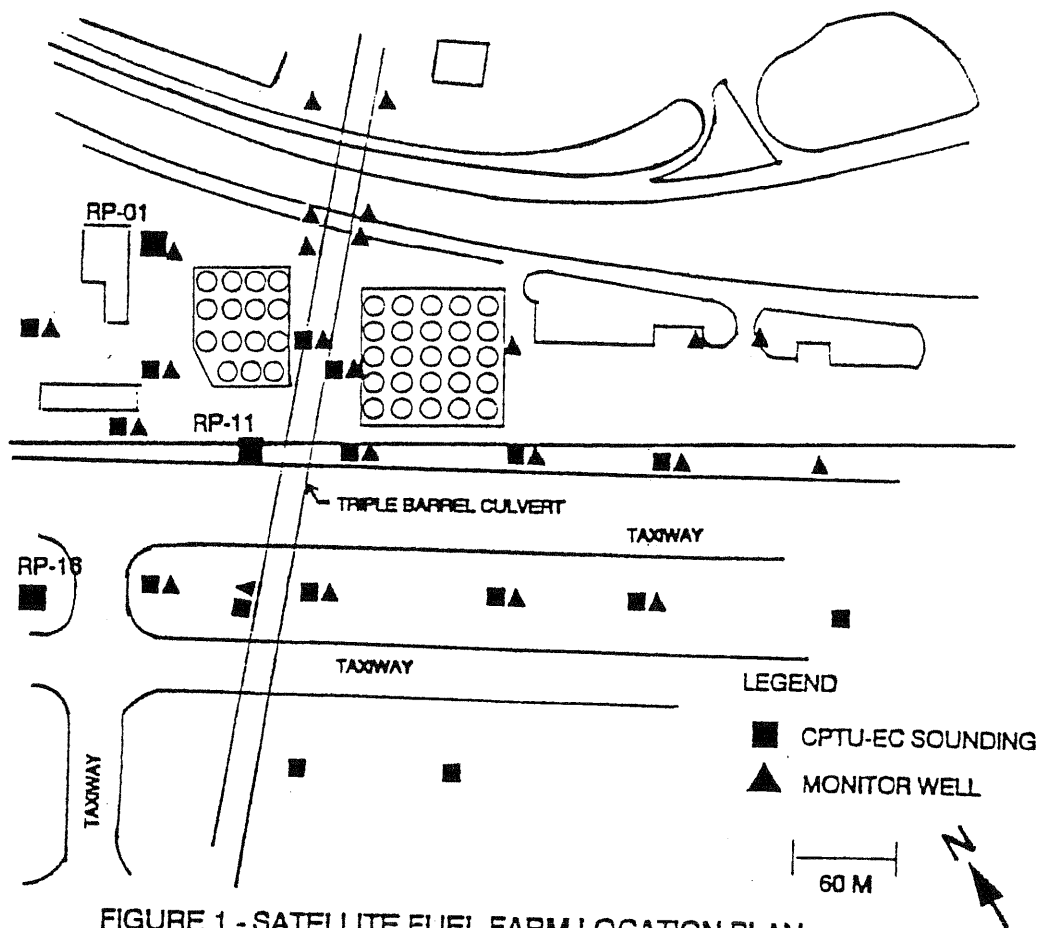


FIGURE 1 - SATELLITE FUEL FARM LOCATION PLAN

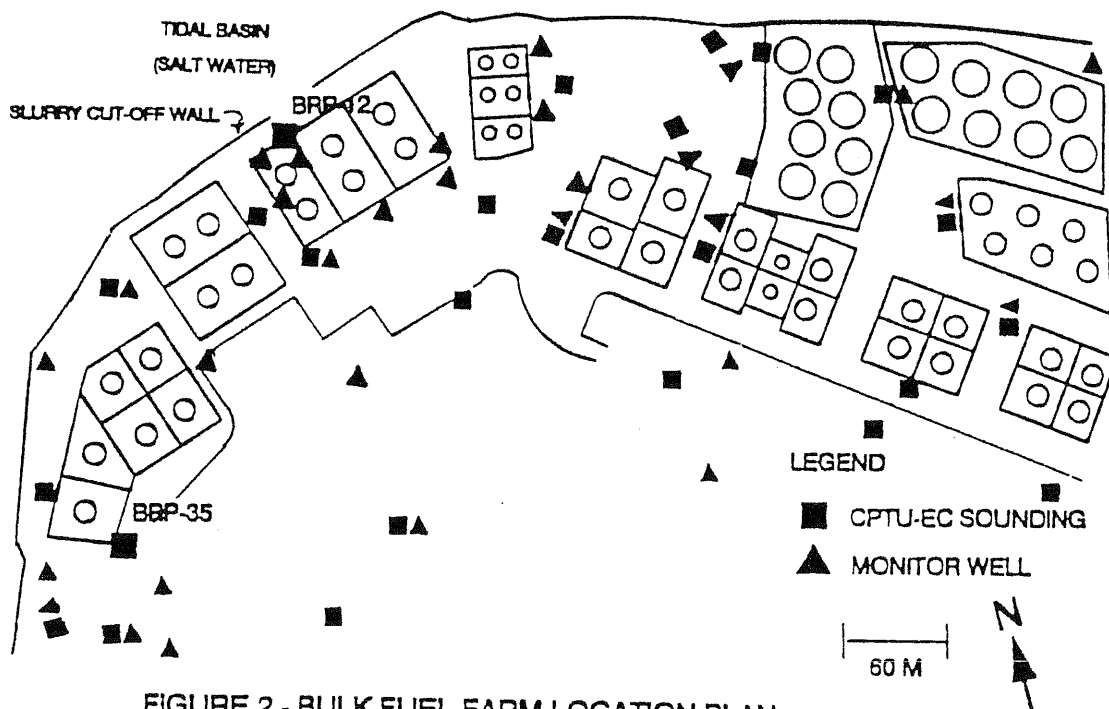


FIGURE 2 - BULK FUEL FARM LOCATION PLAN

Saturation The degree of soil saturation has a pronounced effect on soil electrical conductivity. Conductivity increases with increases in water saturation. Partially saturated sands have low electrical conductivity.

Porosity Soil porosity has an affect on soil electrical conductivity (Reference 2). Less pore fluid is required to fully saturate the pore space of a low porosity (dense) soil, resulting in lower soil electrical conductivity.

LABORATORY PROGRAM

The laboratory program to determine the effects of free phase petroleum hydrocarbon contamination on granular soil electrical conductivity included a total of 60 tests. Samples of soils from JFKIA site excavations, brackish (salty) ground water from site monitor wells, and samples of jet fuel were used to provide a range of variables that might be encountered during field testing. Soil samples were compacted to different porosities (densities) to determine the sensitivity of test results to porosity changes. Over the range of porosities expected to be representative for field conditions, the effects of porosity variation on soil electrical conductivity were considered to be relatively minor when compared to the changes in soil conductivity induced by variation in hydrocarbon content.

Laboratory testing showed that the electrical conductivity of the JFKIA sand samples depended primarily on the amount of water filling the soil pore spaces (degree of water saturation). Soil conductivity decreased with increasing substitution of pore water by jet fuel (Figure 3). A jet fuel saturated sand sample had an electrical conductivity similar to that of a dry sand sample.

The laboratory study indicated that in order to discriminate between dry sands above the water table, and free phase petroleum hydrocarbon product saturated sands below the water table, data on soil saturation was also required. A pore water pressure transducer, used during Piezometric Cone Penetration Testing (CPTU), was added to the CPTU-EC penetrometer to determine soil saturation.

Soil stratigraphy defined by Cone Penetration Test (CPT) measurements can be used to distinguish between the effects of soil type and pore fluid chemistry on measured soil conductivities. Thus, the soil shear resistance measurements of the CPT penetrometer, the piezometric measurement of the CPTU penetrometer, and soil electrical conductivity measurements were all combined in a CPTU-EC penetrometer in order to provide sufficient data to define petroleum hydrocarbon product contamination of saturated granular soils.

PENETROMETER TECHNIQUE

CPTU-EC penetrometer testing consists of smoothly pushing a small diameter (0.044 m - 1.7 inch), instrumented probe (penetrometer) directly into the ground, while a computer data acquisition system displays and records the soil shear resistance, pore water pressure response and soil electrical conductivity during penetration (Figure 4).

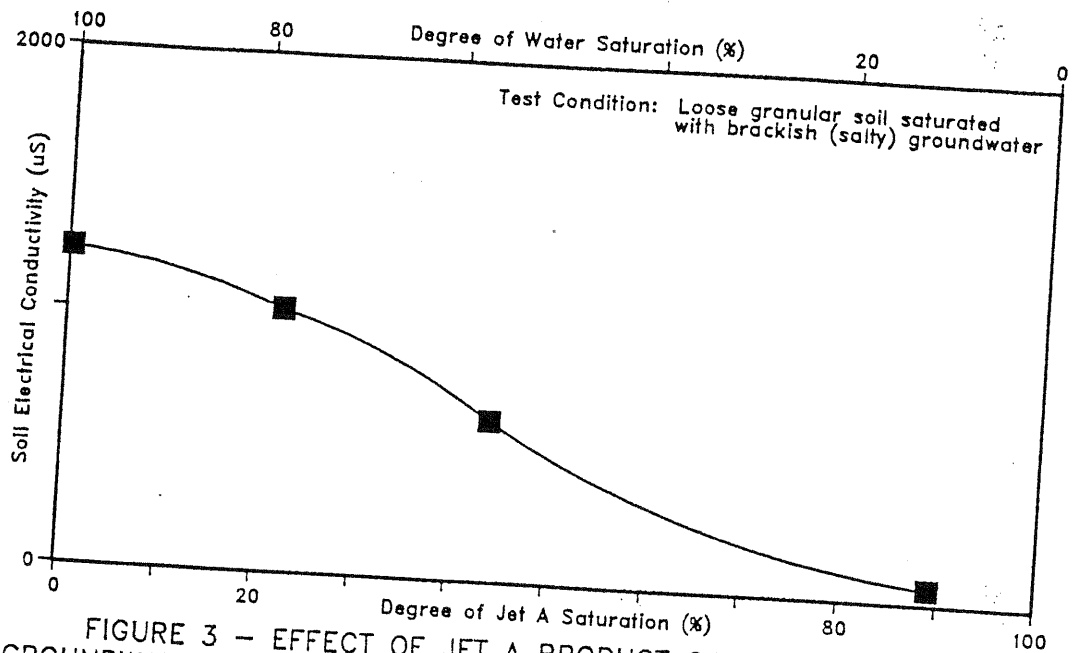


FIGURE 3 - EFFECT OF JET A PRODUCT CONTAMINATION ON GROUNDWATER SATURATED GRANULAR SOIL ELECTRICAL CONDUCTIVITY

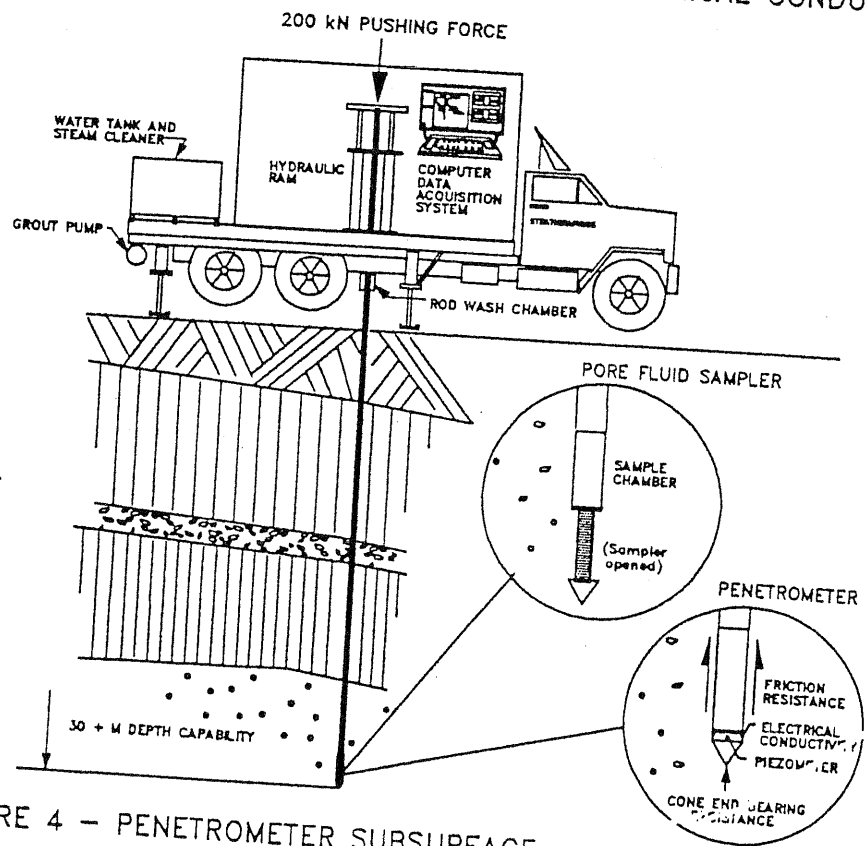


FIGURE 4 - PENETROMETER SUBSURFACE PENETRATION SYSTEM

The penetrometer is mounted at the downhole end of a string of sounding rods. A hydraulic ram is used to smoothly push the penetrometer and rod string directly into the ground, without drilling a borehole, at a constant rate of 0.02 m/sec (4 ft per minute). Electronic signals from downhole sensors inside the penetrometer are transmitted by a cable, strung through the hollow sounding rods, to a data acquisition and display computer system at the surface.

CPTU-EC data are used to develop continuous profiles of geotechnical, hydrogeological, and gross geochemical soil conditions rapidly, accurately and economically. Penetrometer samplers can be used to obtain ground water or soil samples for laboratory testing (Reference 3).

Site disturbance is minimized since no borehole cuttings or drilling fluids are generated during penetrometer operations. Personnel exposure to contaminated soil is significantly less than exposures during drilling and sampling. Penetrometer downhole equipment is easily decontaminated by steam cleaning during retrieval. The small open hole left in soils above the water table after penetrometer retrieval is readily grouted.

CPTU-EC PENETROMETER MEASUREMENTS

The CPTU-EC penetrometer incorporates cone resistance, friction sleeve resistance, piezometric, thermal and soil electrical conductivity sensors. The resistance of a soil to penetration is measured on the tip and along the sides of the CPTU-EC penetrometer. The soil resistance acting on the cone tip is controlled primarily by soil grain size and porosity. The cone resistance measurement has a resolution of about 0.05 to 0.10 m (2 to 4 inches). The sliding friction between the soil and the penetrometer is measured along a sleeve mounted just behind the cone tip. The CPT-EC friction sleeve resistance measurement has a resolution of about 0.15 m (6 inches).

A pressure transducer in the tip of the penetrometer is used to measure the soil pore water pressure response to penetration. Pore water pressure response is primarily controlled by the degree of saturation, potentiometric surface, compressibility and horizontal permeability of the penetrated soil (Reference 4). The CPTU-EC piezometric measurement has a resolution of about 0.03 m (1 inch).

The soil electrical conductivity is measured between two electrodes also mounted in the tip of the CPTU-EC penetrometer. The electrodes are insulated from the steel body of the penetrometer by plastic insulators. The CPT-EC soil electrical conductivity measurement has a resolution of about 0.04 m (1.5 inches). A thermistor inside the CPTU-EC penetrometer provides data on downhole equipment temperatures. These data can be used to adjust the measured soil conductivity to a corrected conductivity at a reference temperature of 25 degrees C.

CPTU-EC data are acquired as analog signals from the transducers inside the penetrometer. The analog signals are transmitted by cable strung through the sounding rod string to a computerized data acquisition system inside the penetrometer truck. The data acquisition system translates the analog signal to a digital value using a 16-bit, analog to digital (A/D) converter. The 16-bit conversion provides a digital data resolution of 1 part in 32,768.

The CPTU-EC data are logged at a 2 Hz frequency. This logging frequency provides insitu soil data at about 0.01 m (3/8 inch) depth intervals. Data appear on a high resolution, color computer monitor in real time. Real time data display allows for the immediate definition of site conditions. Data are logged on hard disk for permanent storage. A preliminary, hard copy sounding log is generated at the conclusion of each test. Recorded data are computer processed to develop interpretations of site conditions.

GENERAL CPTU-EC DATA INTERPRETATION

Correlations between penetrometer data and soil type classifications have been developed from geotechnical soil bearing capacity theory, and observational criteria from adjacent CPT soundings and drilled and sampled boreholes (Reference 5). The CPT cone resistance increases exponentially with increases in soil grain size. The CPT friction ratio (the friction sleeve resistance divided by the cone resistance) increases with increases in the fines content of a soil. A correlation scheme based on the cone resistance and friction ratio values (Figure 5) has proved most useful in interpreting soil types from CPT measurements.

Soil saturation is evaluated using the CPTU-EC piezometric data. Atmospheric (zero) water pore pressure is measured in unsaturated soils. Hydrostatic pore water pressures are generally recorded in high permeability, granular soils below the water table. High pore water pressures are recorded in saturated, fine grained soils during penetrometer advance.

CPTU-EC FIELD TESTING PROGRAM

A total of 48 CPTU-EC soundings were performed at the JFKIA Satellite and Bulk Fuel Farms. The stratigraphy at the two sites is somewhat similar. The surficial soils at both sites consist of a hydraulically placed, fine to medium sand fill, ranging in thickness from about 1.5 to 4.6 m (5 to 15 ft).

At the Satellite Fuel Farm site, this sand fill overlies a discontinuous tidal flat deposit, which consists of about 0 to 1.5 m (0 to 5 ft) of silty clay and peat. At the Bulk Fuel Farm, heterogeneous deposits of refuse and silt interlayer the hydraulic sand fill and tidal flat deposits. Underlying the tidal flat deposits at both sites is a fine to medium sand stratum in excess of 30.5 m (100 ft) thick.

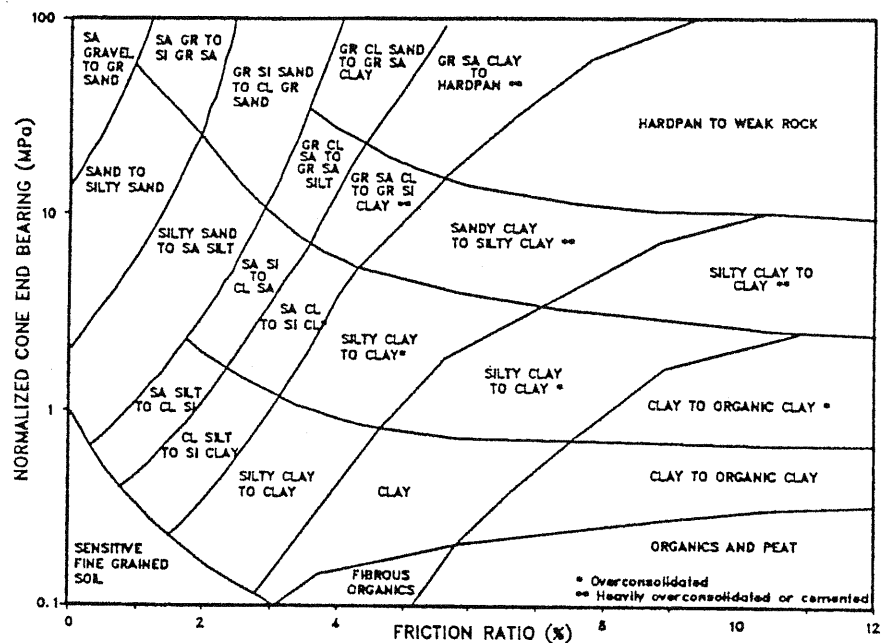


FIGURE 5 - CORRELATION CHART FOR CPT SOIL TYPES

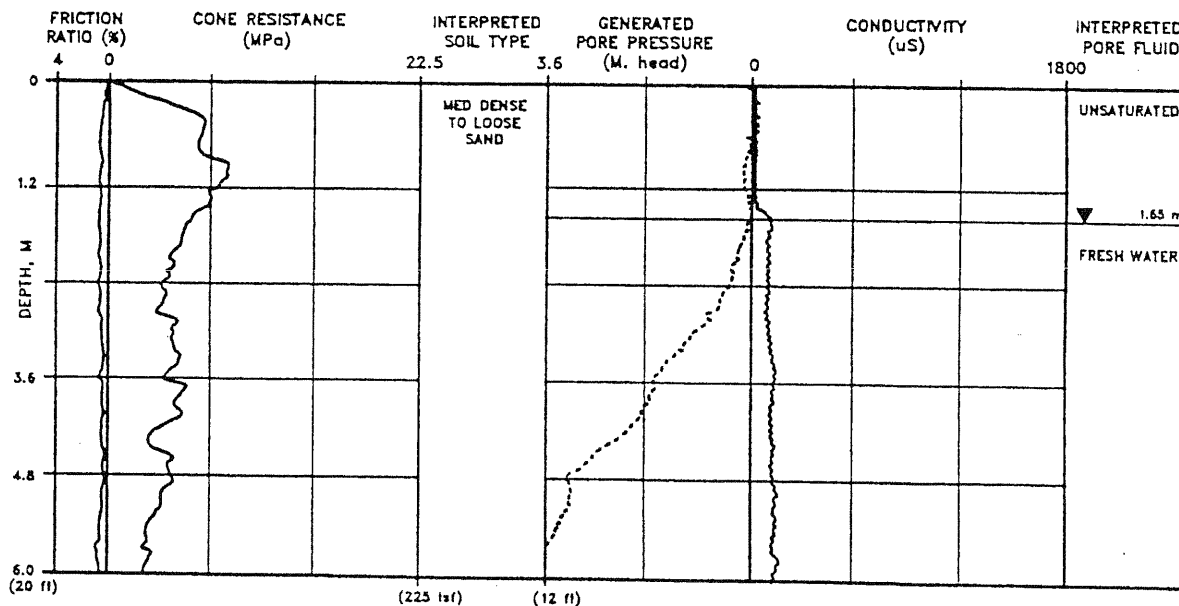


FIGURE 6 - CPTU-EC SOUNDING RP-16

The tidal flat deposits form discontinuous aquitards across the sites, resulting in both locally perched and water table (unconfined) aquifer ground water conditions. At the Bulk Fuel Farm site, the ground water has been partially contained by a slurry cut-off wall. The deeper ground water at both sites is brackish (somewhat salty) with moderate electrical conductivity. Shallow ground water is typically less salty and less conductive, probably reflecting a recent rainwater origin.

The JFKIA Satellite and Bulk Fuel Farms have significant subsurface accumulations of aviation jet fuel, as determined in monitor wells at the two sites. For the Satellite Fuel Farm, free phase petroleum hydrocarbon product thicknesses interpreted from CPTU-EC data were compared to product thicknesses measured in nearby monitor wells. This comparison showed that the general thickness patterns were very consistent, but that the insitu CPTU-EC data indicated product thicknesses to be generally 25 to 50% less than the monitor well product thicknesses.

These results confirm the hypothesis that monitor wells generally contain a thicker accumulation of free phase petroleum hydrocarbon product than is actually present in the soil. This occurs because most products float on the capillary zone above the water table. Thus, the product fills a monitor well for the thickness of the capillary zone and for a depth below the ground water table required to achieve buoyancy equilibrium between the product and ground water.

An uncontaminated, water table (unconfined) aquifer is indicated by the CPTU-EC sounding log at the Satellite Fuel Farm Location RP-16 (Figure 6). The shallow stratigraphy consists of a homogeneous sand stratum. The piezometric measurements indicate the sand to be of medium to high permeability, and indicate a water table at a depth of 1.65 m (5.4 ft).

The soil electrical conductivity increases just above the water table, reflecting increasing soil water content. Soil conductivities are relatively low and constant below the water table, reflecting low ground water salinity conditions. It was subsequently determined that a nearby water main was leaking, and the fresh water leakage was probably responsible for the low soil electrical conductivity measurements.

An accumulation of free phase petroleum hydrocarbon product is indicated by the CPTU-EC sounding log at the Satellite Fuel Farm Location RP-01 (Figure 7). The piezometric measurements indicate a free fluid surface at 1.83 m (6.0 ft) of depth. The very low soil conductivity between 1.83 and 2.10 m (6.0 and 6.9 ft) depths indicates a thin layer of product. Increasing soil conductivity below the product layer indicates increasing ground water salinity and density with depth.

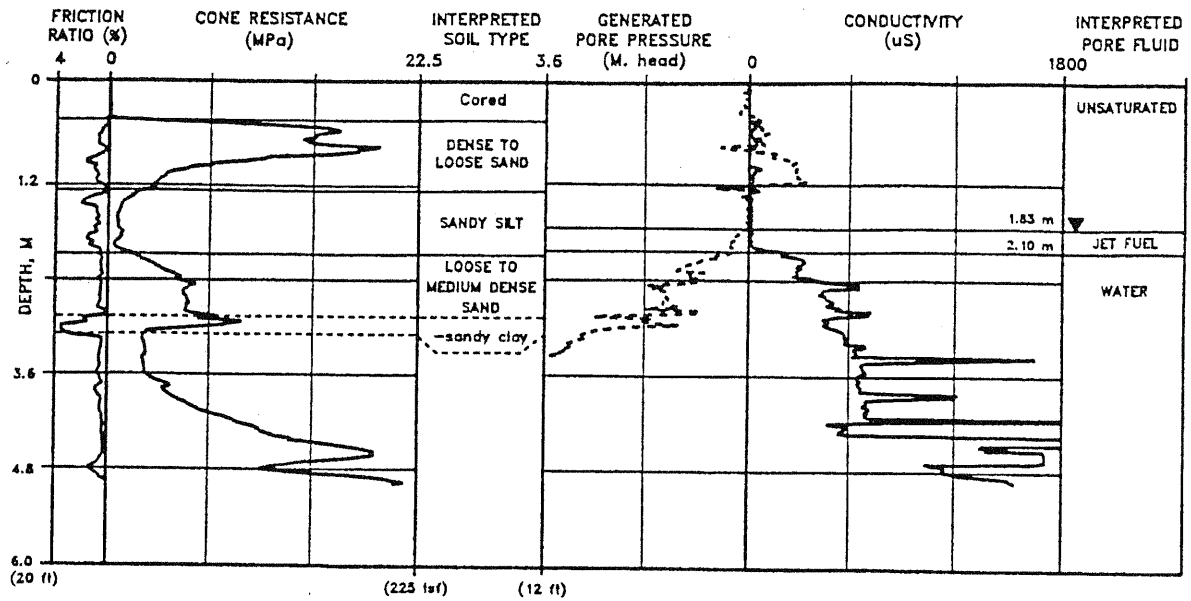


FIGURE 7 - CPTU-EC SOUNDING RP-01

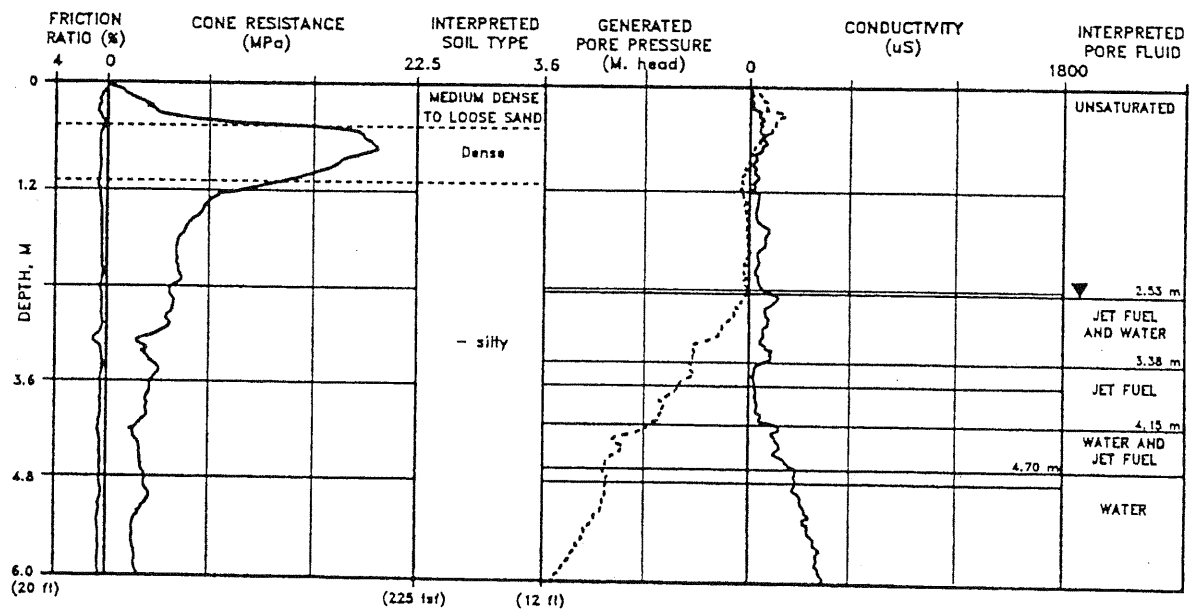


FIGURE 8 - CPTU-EC SOUNDING RP-11

Unusual results were obtained at the Satellite Fuel Farm Location RP-11 (Figure 8) next to a product recovery well. The CPTU-EC data indicate 0.85 m (2.8 ft) of a ground water-petroleum hydrocarbon product mixture, overlying a 0.76 m (2.5 ft) thick layer of product. The product layer overlies another mixed layer, which in turn overlies ground water.

This unexpected sequence is thought to be due to rapidly changing ground water conditions. Record rainfalls during the autumn of 1989 are conjectured to have both raised the locally depressed water table and filled in the cone of depression created by the nearby recovery well. The former surficial product layer became inundated. It is interpreted that due to soil permeability effects, insufficient time had passed prior to the December, 1989 CPTU-EC study for fluid density equilibrium between product and ground water to have been re-established .

This interpretation has been corroborated by CPTU-EC soundings combined with penetrometer ground water sampling at other project sites with similar rapidly changing ground water conditions. A monitor well, typically screened 1.5 m (5 ft) above and 3.0 m (10 ft) below the water table, would provide no hint of this phenomenon, because density equilibrium would occur almost instantaneously in the monitor well riser pipe.

Many of the CPTU-EC sounding logs at the Bulk Fuel Farm were not as definitive as those at the Satellite Fuel Farm. Product thickness trends from the CPTU-EC soundings did generally correspond with monitor well defined trends. However, the presence of perched ground water and product, numerous trapped product lenses, and complex ground water flow conditions caused by the slurry cut-off wall and the discontinuous aquitard, caused CPTU-EC data interpretation to be much more subjective than at the Satellite Fuel Farm.

Ground truthing the CPTU-EC data, acquired at 0.01 m (3/8 inch) intervals to monitor wells screened over 5 m (15 ft) lengths may not be appropriate for the complex site conditions at the Bulk Fuel Farm. The CPTU-EC sounding log at Location BRP-12 (Figure 9) illustrates some of the difficulties in interpreting data at sites with a complex hydrostratigraphy.

The CPTU-EC results were definitive in areas of the Bulk Fuel Farm where uniform conditions existed. The presence of free phase petroleum hydrocarbon product overlying a water table aquifer is indicated by the CPTU-EC sounding log at Location BRP-35 (Figure 10). Soil electrical conductivity is very low between the free fluid surface at a depth of 2.80 m (9.2 ft) and a depth of 3.29 m (10.8 ft), indicating a 0.49 m (1.6 ft) thick layer of product. A 0.15 m (0.5 ft) thick transition zone underlies the product layer and probably consists of soil saturated with both product and ground water.

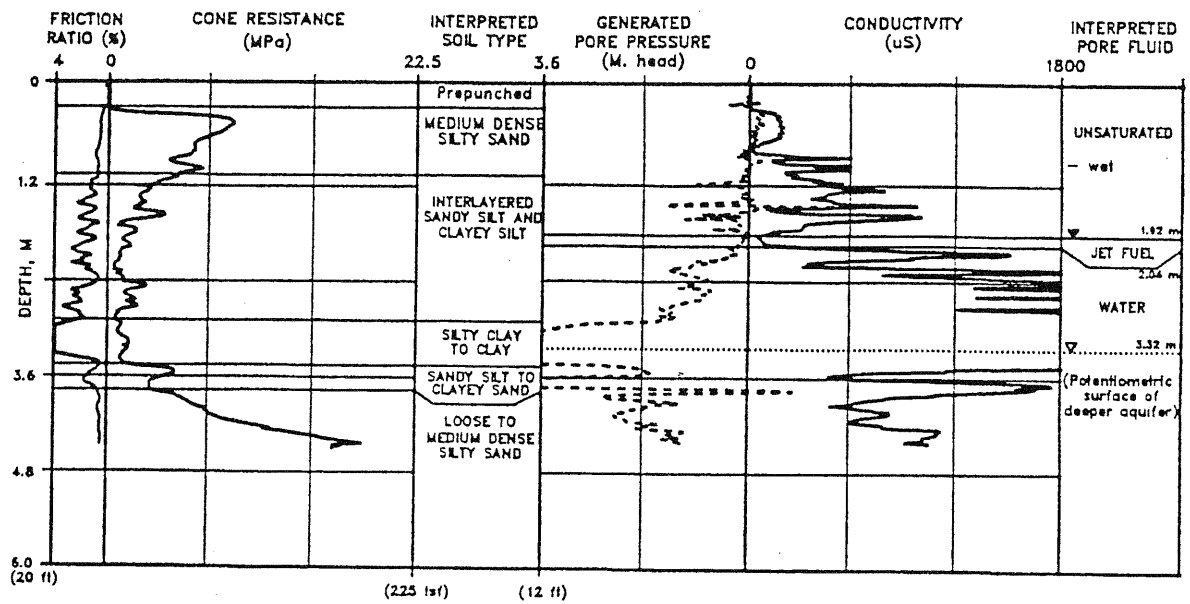


FIGURE 9 - CPTU-EC SOUNDING BRP-12

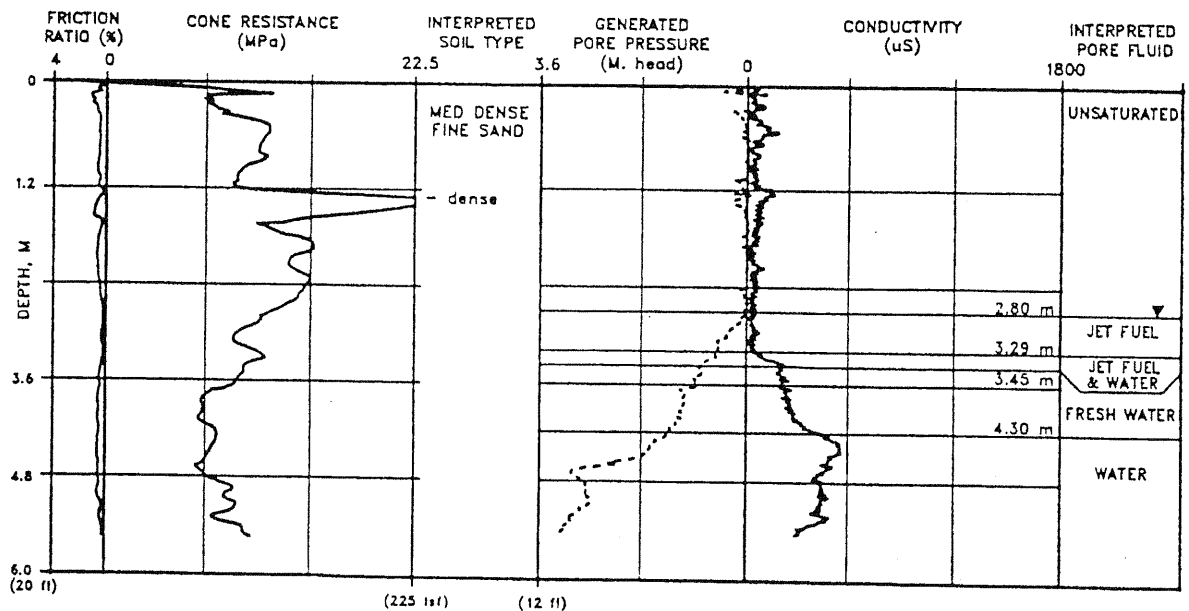


FIGURE 10 - CPTU-EC SOUNDING BRP-35

CPTU-EC COSTS AND PRODUCTIVITY

A comparison of production rates and costs for CPTU-EC and conventional monitor well surveys is as follows:

	<u>CPTU-EC</u>	<u>Monitor Wells</u>
Production Rate	8 to 12/day	1 to 2/day
Unit Cost	\$66/m (\$20/ft)	\$131/m (\$40/ft)
Cost per location, 6.1 m (20 ft) depth	\$485/ea.*	\$1066/ea.*

* includes data interpretation or inspection.

The CPTU-EC method provides a more rapid means of surveying an area, and is less than one half the cost of conventional monitor well survey methods on a per location basis.

CONCLUSIONS

The CPTU-EC penetrometer method has been shown to provide a rapid means of surveying sand aquifers for free phase petroleum hydrocarbon product contamination. In areas of more complex stratigraphy, additional testing is necessary to verify the applicability of CPTU-EC methods. Monitor wells with long screened lengths may not provide the best method of ground truthing CPTU-EC measurements at sites with complex hydrostratigraphic conditions.

Penetrometer ground water sampling should be included in CPTU-EC field investigation programs to provide direct samples of CPTU-EC identified anomalous ground water zones. Sensitive CPTU-EC piezometric transducers should be used to provide high accuracy in water table location.

The rapidity and the relative non-destructive nature of the CPTU-EC method especially provides advantages in areas of high priority usage or sensitivity, such as active apron areas of airport terminals, or in residential areas surrounding contamination sources. The CPTU-EC method in many cases, allows for more rapid and better definition of the true thickness of free phase petroleum hydrocarbon products in ground water. Cost savings in initial survey work should translate into better placement of permanent monitor and recovery wells, resulting in decreased overall remediation/investigation program costs.

REFERENCES

- [1] Strutynsky, A.I., B.J. Douglas, L.J. Mahar, G.F. Edmonds, and E. Hincey, 1985. Arctic Penetration Tests Systems. Civil Engineering in the Arctic Offshore, ASCE, pp 162-168.
- [2] Kutter, K.L., K. Arulanandan, and Y.F. Dafalias, 1979. A Comparison of Electrical and Penetration Methods of Site Investigation. Offshore Technology Conference Proceedings.
- [3] Strutynsky, A.I., T.J. Sainey, 1991. Use of Piezometric Cone Penetration Testing and Penetrometer Ground Water Sampling for Volatile Organic Contaminant Plume Detection. Petroleum Hydrocarbons and Organic Chemicals in Ground Water. API/NWWA, Conference Proceedings, pp 71-84.
- [4] Saines, M., A.I. Strutynsky, and G. Lytwynyshyn, 1989. Use of Piezometric Cone Penetration Testing in Hydrogeologic Investigations. First USA/USSR Hydrogeology Conference, Moscow, USSR, NWWA.
- [5] Douglas, B.J., R.S. Olsen, 1981. Soil Classification using the Electric Cone Penetrometer. Cone Penetration Testing and Experience, Conference Proceedings, ASCE, pp 209-227.

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USING INNOVATIVE SAMPLE COLLECTION AND FIELD ANALYTICAL TECHNIQUES TO RAPIDLY CHARACTERIZE MULTIPLE PETROLEUM HYDROCARBON SITES

ABSTRACT

Innovative exploration methods and advanced on-site field analytical techniques identified the presence or absence of petroleum hydrocarbon constituents (PHCs) in the soil and groundwater at multiple sites, quickly and effectively. Where present, the vertical and horizontal extent of PHCs was identified. The field program was completed using two terraprobe systems, a cone penetrometer unit, and two mobile laboratories in a four-week time frame.

Piezometric Cone Penetrometer Tests with soil Electrical Conductivity (CPTU-EC) measurements were used to rapidly assess site hydrostratigraphy. Three CPTU-EC soundings were typically performed at each site. Aquifer and aquitard units were identified in real time. Soil and electrical conductivity measurements were used to evaluate whether free-phase products were

present at each site. Small diameter piezometers were installed in each CPTU-EC hole to identify the direction of ground water flow and to allow for sampling of ground water. Penetrometer soil and ground water sampling were performed when lighter weight geoprobe units could not penetrate to desired depths. The penetrometer truck was able to complete exploration work at one and one-half sites per day, on average.

Using a phased approach, soil and ground water samples were analyzed for PHCs in the field, providing quality real-time data. During Phase I, all soil samples were analyzed for total petroleum hydrocarbons (TPH) by infrared (IR) spectroscopy, using a modification of USEPA Method 418.1. Data from these initial locations directed subsequent soil and ground water sampling, both horizontally and vertically. Approximately 30 percent of the samples underwent subsequent analysis (Phase II)

using a gas chromatograph (GC) for the specific analyte determination of benzene, toluene, ethylbenzene, and xylenes (BTEX), and fuel fingerprint identification.

INTRODUCTION

Corporate policy requires environmental assessment of all properties considered for acquisition or divestiture. One objective of these assessments is to determine the extent, if any, to which soil and ground water have been impacted by past operations. The knowledge is required to determine financial exposure and environmental obligations.

In 1992, approximately 400 miles of product pipeline in northern Illinois, Indiana, and western Ohio was investigated. Environmental assessments were required at seven pumping stations and three terminals associated with this pipeline. In addition, eight old spill sites at various locations along the pipeline right-of-way were investigated. Table 1 is a summary of site activities. Timing associated with negotiations and the desired close-date resulted in a 12-week window in which to complete the environmental assessments.

In response to a request for proposal, proposals ranging for \$600,000 to \$1,200,000 were received. The scope of work addressed in these proposals included the determination of the presence or absence of PHCs in soil and ground water and, if present, the vertical and horizontal extent. Both traditional (soil borings, monitoring wells, and off-site laboratory analyses) and innovative

methods were proposed. Traditional assessment methods were rejected for the following reasons:

1. Time constraints did not allow the use of slower methods.
2. Data quality objectives did not require USEPA Level IV data.
3. Cost.

The plan that proposed the use of a cone penetrometer truck (CPT), two geoprobe units and field based analytical procedures was selected for the following reasons:

1. CPT techniques offered acquisition of stratigraphic and hydrologic data.
2. Geoprobe soil and ground water sample collection methods offered the collection of a large number of samples in a short amount of time.
3. Use of the CPTU-EC and geoprobe did not result in the generation of soil cuttings and monitoring well purge water requiring off-site disposal.
4. The field analytical plan best suited corporate requirements and was backed with directly relevant experience.
5. The proposed project schedule was credible and met corporate requirements.
6. Use of the proposed innovative techniques substantially reduced the cost of the project.

TABLE 1
Project Summary

Number of Sites	Total Number of Explorations	Total Number of Samples Collected	Total Number of Analyses ¹
18	575	913	1,498

NOTES:

- ¹ The total number of analysis includes quality control samples and multiple analysis of a single sample.

TECHNICAL APPROACH

Three key elements used in the alternate approach to the 18 site investigations were CPT, geoprobe, and field analysis. This combination of innovative collection and on-site analytical techniques enabled the subsurface investigations of each site to be quick, thorough, and cost-effective.

Cone Penetrometer. The penetrometer technique provides various advantages during geo-environmental studies. These include a relatively nondestructive test procedure generating no cuttings or drilling fluids; real-time, computerized data acquisition and presentation; continuous profiling capability; minimal exposure of personnel to possible contaminants; and higher productivity with lower cost as compared to borehole techniques.

CPTU-EC soundings were performed at several locations within each study area. On-site personnel used these soundings to investigate stratigraphy, to determine the depth to ground water, and to identify soil sample collection depths. Each CPTU-EC sounding generated physical data about subsurface soil conditions interpreted from

cone tip and friction sleeve-bearing resistance, generated pore pressure, and electrical conductivity.

CPTU-EC penetrometer testing consisted of using a large hydraulic ram to push a small diameter (1.7-inch) instrumented probe (penetrometer) into the ground while measuring the soil shear resistance to penetration. In addition to shear resistance, a pressure transducer and a soil electrical conductivity sensor were mounted on the cone tip. The pressure transducer was used to measure the water pressure response of the saturated soil. Time-dependent measures (dissipation tests) were used to calculate in situ hydraulic conductivities. Soil electrical conductivity primarily depends on pore fluid chemistry (identifying possible locations of nonconductive PHC layers¹), soil saturation, and clay content. Downhole sensors transmitted electrical signals to an uphole computer for real-time display allowing immediate interpretation of subsurface conditions. Both soil and ground water samples could be collected by CPT techniques² when depths exceeded geoprobe capabilities.

The CPTU-EC data were reviewed before sample collection activities, and used to locate subsequent subsurface soil collection depths. Information about soil saturation and electrical conductivity was of particular interest for indicating the probable whereabouts of subsurface PHCs.

Piezometer Installation. Several small diameter (3/4-inch nominal) polyvinyl chloride (PVC) piezometers were installed at each site. All piezometers were screened across the water table in an attempt to intercept any hydrocarbon layer that might be present. The 15-foot-long slotted PVC screen was covered with a geofabric before installation to reduce infiltration of fines.

At each piezometer location, the CPT rig pushed a steel casing to the required depth (approximately 20 feet below ground surface [bgs]). The PVC screen and riser were placed inside the casing, which was then retracted leaving the piezometer material below the water table. The piezometer installation was completed with a protective casing to minimize tampering. All downhole equipment and PVC piezometer materials were steam-cleaned prior to installation.

All penetrometer downhole equipment was automatically decontaminated during retrieval by passing it through a sealed rodwashing chamber, mounted below the hydraulic ram, and connected to the onboard steam cleaner. The open hole was grouted using a proprietary system that continuously pumped grout into the annular space.

Geoprobe. The geoprobe is a hydraulic pushing system mounted on a truck that

can push 1.25-inch sampling rods to a depth of approximately 30 feet bgs. By using the geoprobe system to collect soil and ground water samples, a reduction in time and cost was realized. Each geoprobe unit was able to collect 10 to 15 soil samples a day from 8 to 10 explorations. Additionally, this technique did not generate soil cuttings. The geoprobe unit is relatively small and lightweight compared to traditional drill rigs making it easier to access congested terminal locations.

Initial subsurface sampling locations were sited next to potential source locations; subsequent explorations were placed downgradient from the potential sources. The specific locations for subsequent sampling were based on information obtained during the initial sample collection and analytical data from the mobile laboratory. By directing the exploration program in the field, only necessary samples were collected and analyzed. This reduced the number of samples necessary to characterize the site. Generally, soils were collected from strata exhibiting depressed electrical conductivity and from the area around the water table.

DATA QUALITY OBJECTIVES

Data generated during field activities were characterized as USEPA Level I and Level II (USEPA, 1987).

With respect to the pipeline project, Level I data were qualitative and semi-quantitative and provided information on the presence or absence of contamination. Data included measurements from handheld photoionization detectors, pH meters, and specific conductivity probes. Level II data

were those data generated by an on-site laboratory and were both qualitative or quantitative. Level II data collected during this investigation included TPH by IR detection, volatile organic compounds (VOCs) by GC (mainly BTEX), and petroleum fingerprints using GC.

FIELD ANALYTICAL METHODS

The detection of fuel-related hydrocarbons in site soils and ground water was performed using a combination of two analytical techniques: IR spectroscopy and gas chromatography. All soil samples were analyzed for TPH by IR; approximately 30 percent of the samples were also analyzed using GC.

Total Petroleum Hydrocarbons by Infrared Analysis. TPH were analyzed by IR spectroscopy as outlined in USEPA Method 418.1. The modified procedure³ combined solvent extraction and silica gel cleanup into a rapid single step micro-extraction that uses smaller volumes of both solvent and sample. As many as 90 samples were analyzed in a single day by the two field operators. The analytical detection limit for this modified procedure is 50 milligrams per kilogram (mg/kg).

Gas Chromatography Analysis. GC was used for the specific determination of individual volatile aromatic hydrocarbons including BTEX⁴, and to generate fuel "fingerprints" for selected mixtures of fuels through Carbon-28 (C₂₈). Generated fuel fingerprints were compared to known fuel products for both identification and quantitation and interpreted against current standards described in ASTM Method D 3328-78. The detection limit for the individual target fuel components

in soil was one to two mg/kg, while the fuel mixtures had detection limits ranging from 10 to 100 mg/kg.

Quality Control. Quality Control for the aforementioned analytical methods was performed to a degree sufficient to evaluate data precision and accuracy and system performance while not inhibiting the ability to generate real-time data and maintain high sample throughput.

COMPARATIVE ANALYSIS

Tables 2 and 3 contrast and compare a traditional approach to site investigations (i.e., drilling and off-site laboratory analysis) with the innovative approach used during this program. Through the use of innovative collection and analytical techniques, the overall cost and schedule of the program was reduced without compromising the data quality needed to meet project objectives.

Impact On Program. Table 2 presents the actual cost incurred by the innovative program. Table 3 is an estimate of the costs that would have been incurred if a more traditional approach had been followed. Table 4 is a comparison of the two methods illustrating the cost savings for each unit item. However, possibly the greatest savings is much more difficult to measure - is the reduction in cycle-time that shortened the schedule significantly.

CASE STUDY

Investigation results for a terminal of approximately 37 acres is presented as a case study to illustrate the cost savings. The on-site review and historical data

TABLE 2
Project Cost Summary
Innovative Approach

	No. of Explorations	No. of Samples	Mob/ Demob ¹	Average Unit Cost	No. of Days	Total Cost
Geoprobe	508	757	\$5,000	\$335/ Exploration	43	\$170,180 ³
CPT	64	53	\$2,200	\$1,471/ Exploration	19	\$94,150 ⁴
Field Analysis	--	1,498 ²	\$46,900	\$151/Analysis	43	\$226,198 ⁵
Drilling	3	0	\$1,600	\$13,850/Well	8	\$41,550 ⁶
Reporting	18 Reports	--	--	\$5,933/Report	16	\$106,796
Project Management	--	--	--	\$2,931/Site	70	\$52,758
Totals	575	--	\$55,700	--	--	\$691,632
Average Cost per Site - \$38,456						

Notes:

¹ This includes the cost of staging, moving equipment and personnel to and from all locations.

² Includes samples collected by other means (i.e., surface soils, sediments, surface water, and a smaller number of quality control samples).

³ Cost includes all labor (2-person crew).

⁴ Cost includes 51, 3/4-inch piezometer and all labor (2-person crew plus 1 inspector).

⁵ Cost includes all labor (2-person crew).

⁶ Cost includes three 4-inch PVC monitoring wells, and labor for a 2-person crew plus 1 inspector (monitoring wells were necessary because of a very thick gravel deposit (approximately 45 feet of gravel to the water table)).

TABLE 3
Estimated Project Cost Summary
Traditional Approach

	Total No. of Explorations/Analysis	Mob/ Demob	Est. Cost Per Unit	Subtotal Cost	Est. No. of Drums	Estimated Cost for Drum Disposal	Total Cost
Soil Boring ¹	508	\$1,750	\$600 ⁴ / Boring	\$306,550	127 soil ²	\$209,550	\$516,100
Monitoring Well Installation	54	\$2,200	\$1,815 ⁵ / Monitoring Well	\$92,565 Plus \$41,550 three gravel wells see Table 1	14 soil ² Plus 27 drums of purge water ³	\$31,065	\$165,180
Off-Site Laboratory Analysis	1,498 Analyses IR - 806 BTEX plus Finger Print - 346	\$4,300 Shipping Cost	IR - \$180 BTEX plus Fuel Finger Print - \$600 ⁶	IR - \$145,080 BTEX plus Fuel Finger Print - \$207,600	—	—	\$356,980
Reporting ⁷	18 Reports	—	—	\$5,933/ Report	—	—	\$106,794
Project Management ⁷	—	—	—	\$2,931/ Site	—	—	\$52,758
Totals	562 Explorations; 1,498 Analysis	—	—	—	168	\$240,615	\$1,197,812

Average Cost per Site - \$94,635

Notes:

¹ Average depth of soil exploration from Table 1 is 12 feet.

² Drill cuttings disposal. Assume one soil drum for every 4 soil borings requires off-site disposal and analysis: RCRA Characteristics \$210 [reactivity \$130, corrosivity \$15, ignitability \$65], TCLP analysis at \$1,200, TPH at \$60, Total BTEX at \$100, trucking at \$35/drum and bioremediation at \$45/drum. Total cost per soil drum estimated at \$1,650.

³ Purge water disposal. Assume one drum of purge water for every four wells requires off-site disposal. Analysis required: TPH at \$60, Total BTEX at \$100, Total BTEX at \$100, and trucking at \$35/drum, and treatment at \$100/drum. Total estimated cost per purge water drum at \$295.

⁴ Assume 4.25-inch hollow stem augers (HSA) with 5-foot sampling. Cost estimated at \$35/foot and bentonite grout at \$15/foot combined estimated cost of \$50/foot. Labor cost included in footage rate.

⁵ Assumed 6.25-inch HSA 5-foot sampling at \$35/foot and well material (4-inch PVC riser, \$6/foot; 15-foot PVC screen, \$32/foot; grout, \$15/foot; protective casing \$250 each) estimated cost per 25-foot installation \$1,815. Labor cost included in footage rate.

⁶ Assume 24-hour turnaround is 3 times the cost of non-expedited analysis.

⁷ Cost for Reporting and Project Management assumed to be the same as presented in Table 2.

Table 4
Cost Summary
Innovative Compared To Traditional Approach

	Average Cost Per Site	Average Cost Per Exploration	Average Cost Per Analysis	No. Of Days	Totals
Innovative Approach	\$38,456	\$532 ¹	\$151 ²	70	\$692,223
Traditional Approach	\$94,655	\$1,185 ³	\$238 ⁴	120	\$1,197,812
Savings Using Innovative Approach	\$56,199	\$653	\$87	50	\$505,589

Notes:

- ¹ Average cost per exploration includes cost of geoprobe, CPT, and Drilling from Table 2 for 575 explorations.
- ² Average cost per sample includes IR, BTEX, and fuel fingerprinting from Table 2 for 1,498 analysis.
- ³ Average cost per exploration includes cost for Soil Borings and Monitoring Well Installation from Table 3 for 575 explorations.
- ⁴ Average cost per sample includes IR, BTEX, and fuel fingerprinting from Table 3 for 1,498 analysis.

provided indicated that the pipeline manifold/sump area, the large aboveground breakout tanks, the railroad and truck-loading racks, and the warehouse were potential sources. The soil sampling and piezometer locations were placed using this information (Figure 1). Physical and chemical data from soils and ground water were collected at the Terminal site using a combination of cone penetrometer and geoprobe system sample collection techniques. All samples were analyzed using a combination of IR spectroscopy and GC. This work was completed including sample analysis and preliminary data evaluation in approximately five geoprobe/field laboratory days and two CPT days.

Stratigraphy and Hydrogeology. The terminal is located on a small hill. The large aboveground breakout tanks are located at the top of the hill, whereas the manifold area, loading racks and

warehouse are at the bottom of the hill. The soils are interbedded sands, silts, and clays (Figure 2). There are two aquifers below the site, a perched system below the breakout tanks, and a regional sand and gravel aquifer.

Data Interpretation. Eighty-six soil, ten groundwater, five sediment, and four surface water samples were collected and analyzed for the presence and quantitation of petroleum hydrocarbons. PHCs were found in 44 of the soil samples associated with the loading racks and east of the large aboveground breakout tanks (see Figure 1). Sediment samples collected along the site boundary contained detectable concentrations of PHCs. No PHCs were reported in any surface waters associated with these sediment samples. Four of the ten ground water samples collected contained PHCs.

The apparently upgradient ground water sample (PW-0709) contained PHCs identified as weathered gasoline at 0.34 milligrams per liter (mg/L). The source area for these compounds was undetermined, and may have been related to an unidentified off-site source. The two ground water samples (PZ-0706 and PW-0707) collected downgradient from the bermed area (large aboveground breakout tanks) contained BTEX and a fuel fingerprint identified as gasoline at 9.4 and 83 mg/L, respectively. Gasoline was also identified in the ground water sample collected from the upper perched system (PZ-0704) at 0.94 mg/L (see Figure 2).

The soils data suggest that the bermed area, the manifold/sump area, and the two loading racks were probable source areas for the hydrocarbons present. The presence of PHCs in soils adjacent to the site boundaries and in the ground water suggested that PHCs may have migrated beyond the terminal boundary. Additionally, it appeared that PHCs may also have been migrating onto the site from an unidentified source area.

RESULTS

The results of applying the innovative techniques described in this paper are as follows:

1. Corporate environmental assessment requirements were fulfilled and meet the negotiated divestiture time schedule.
2. A total of 18 sites (seven pumping stations, three terminals, and eight spill sites) were assessed in four weeks of field

work. Total project duration was 12 weeks.

3. The concentration and extent of PHCs in soil and ground water were mapped for each spill site. The concentration and extent of PHCs in soil and ground water mapping did not extend beyond property boundaries.

4. Environmental assessment costs were substantially reduced by utilizing innovative rather than traditional techniques.

CONCLUSIONS

By utilizing a cone penetrometer truck, two geoprobe units and field analytical procedures, the concentration of petroleum hydrocarbon constituents in soil and ground water was determined at 18 sites in a short time period and at reduced price. The extent of petroleum hydrocarbons in soil and ground water was mapped for each site. Mapping was limited to property boundaries for pumping stations and terminals. The successful completion of the environmental assessment fulfilled corporate requirements and met the negotiation divestiture time table.

ACKNOWLEDGEMENTS

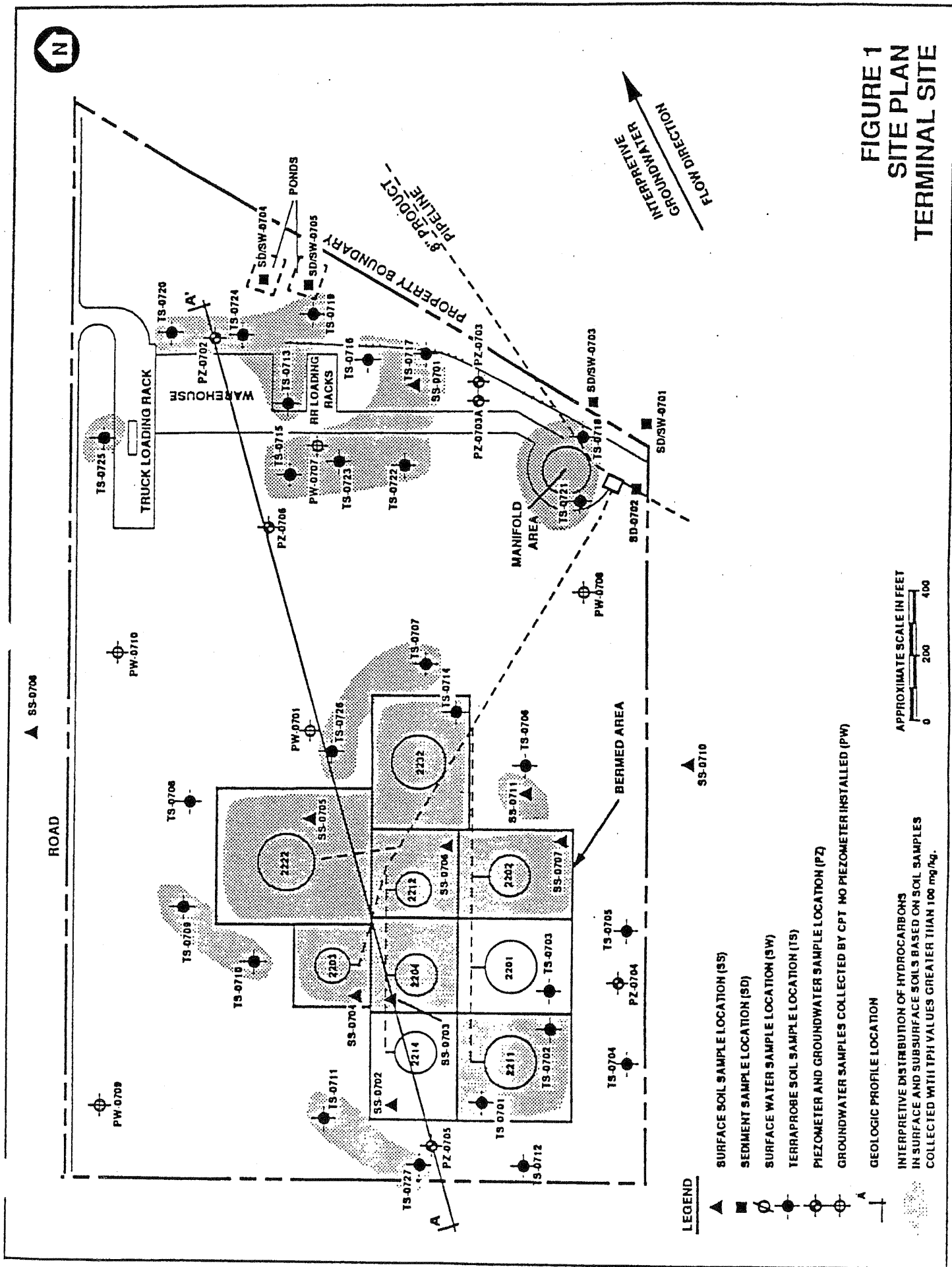
ARCO Pipe Line Company acknowledges ABB Environmental Services and Stratographics Geotechnical Data Acquisition Corporation for their professional and exemplary conduct during this project. In addition, the authors

thank their respective companies for the opportunity to publish this work.

REFERENCES

1. Strutytsky, A.I.; R. Sandiford, D. Cavaliere, 1991. "Use of Piezometric Cone Penetration Testing with Electrical Conductivity Measurements (CPTU-EC) for Detection of Hydrocarbon Contamination in Saturated Granular Soils. Ground Water and Vadose Zone Investigations." ASTM.
2. Strutytsky, A.I.; T. Sainey, 1990. "Use of the Piezometric Cone Penetration Test and Penetrometer Groundwater Sampling for Volatile Organic Contaminant Plume Detection.", Paper presented at the 1990 API/NWWA Petroleum Hydrocarbons and Organic Chemicals in Ground Water: Prevention, Detection, and Restoration Conference, Houston, Texas. November.
3. Kasper, K.; D. Twomey and D. Dinsmore, 1992. "On-Site Analysis of Fuel-Related Hydrocarbons In Soils By Infrared Methods." Paper presented at 1991 NWWA Petroleum Hydrocarbons and Organic Chemicals in Ground Water: Prevention, Detection, and Restoration Conference, Houston, Texas. November 20 - 21.
4. Turner, S. A.; T. L. Francoeur, D.M. Twomey, B. K. Butler, 1991. "On-Site Analysis of Chlorinated Solvents in Groundwater by Purge-and-Trap GC". Proceedings from the Second International Symposium of Field Screening Methods for Hazardous Wastes and Toxic Chemicals. February, 1991.

FIGURE 1 SITE PLAN TERMINAL SITE



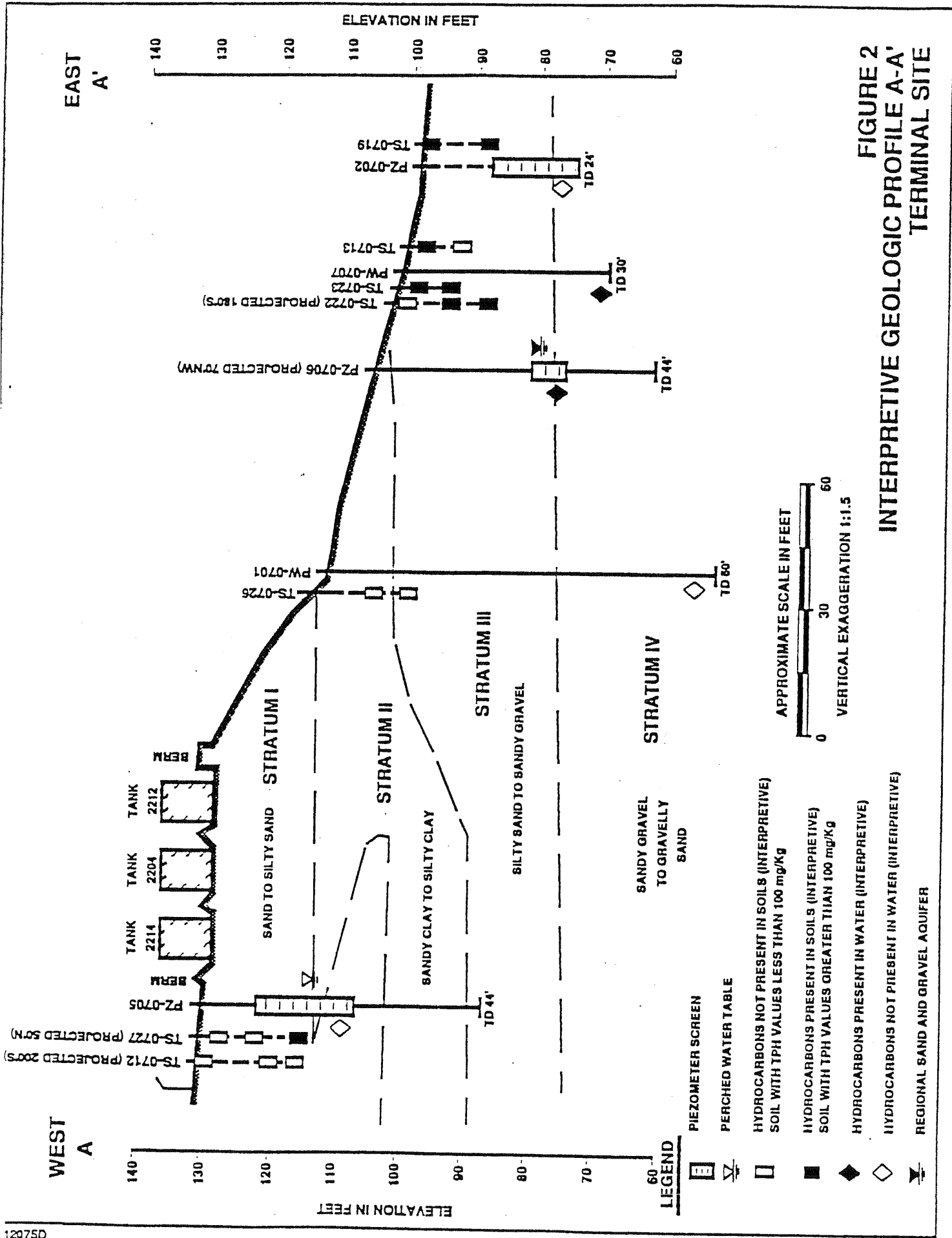


FIGURE 2
INTERPRETIVE GEOLOGIC PROFILE A-A'
TERMINAL SITE

USE OF PIEZOMETRIC CONE PENETRATION TESTING AND PENETROMETER GROUNDWATER SAMPLING FOR VOLATILE ORGANIC CONTAMINANT PLUME DETECTION

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ABSTRACT

Piezometric Cone Penetration Testing (CPTU) and penetrometer groundwater sampling were used in locating a volatile organic contaminant plume at an industrial site in southern Ohio. Nine CPTU tests (soundings) were performed to determine site hydrostratigraphy in real-time. On-site chemical analysis of penetrometer groundwater samples provided near real-time detection of contaminants. These results were used to define subsequent exploration points. Using this investigation approach, drilling operations to set monitor wells began as penetrometer exploration ended. Program quality increased, while exploration costs decreased by using this combination penetrometer-drilling rig approach.

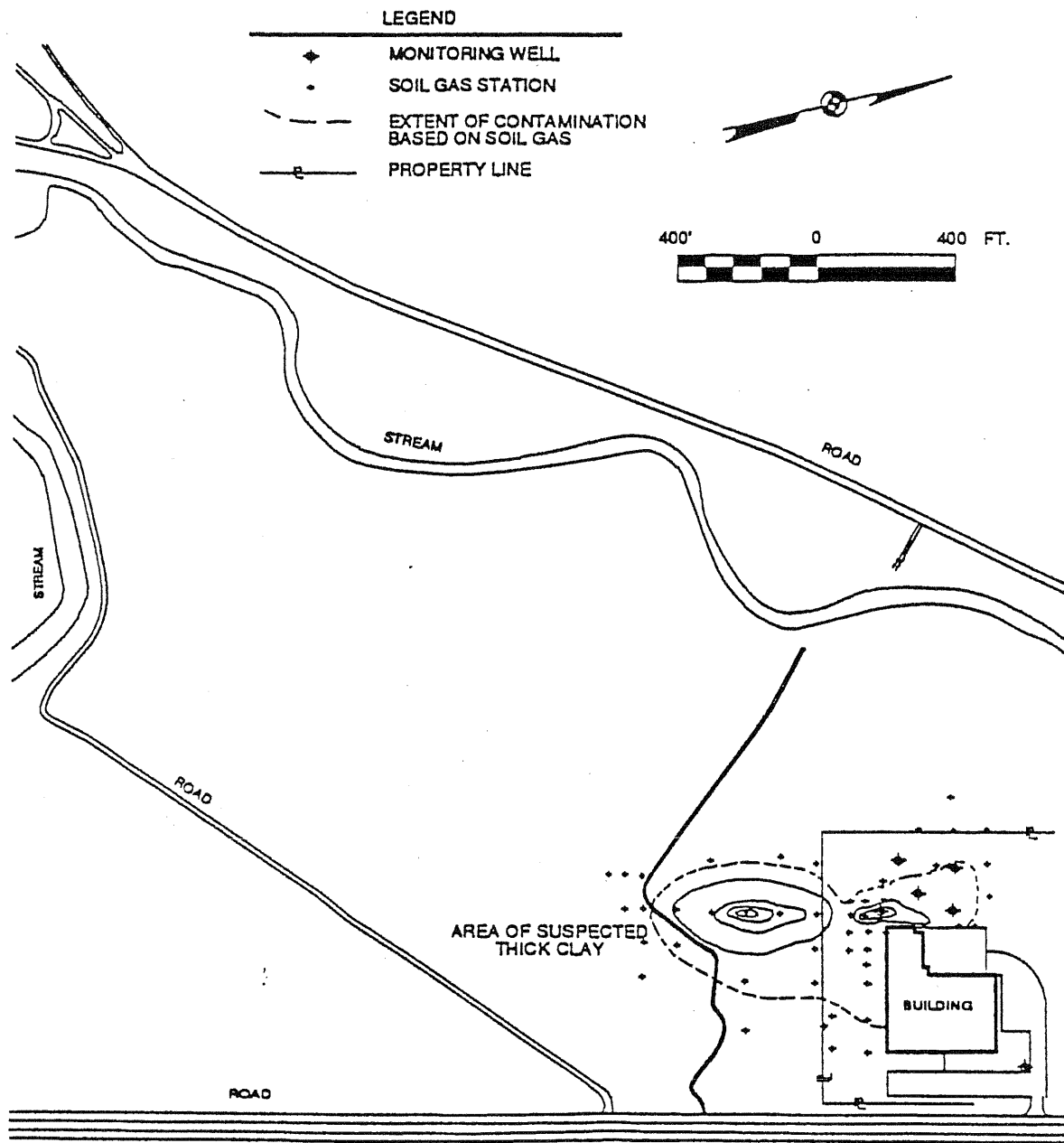
INTRODUCTION

Trichloroethene (TCE), an industrial solvent, along with other volatile organic compounds, was identified in the groundwater at a manufacturing site in rural southern Ohio. Contamination was initially found in four monitor wells, located around a suspected source. The immediate soils consist of mixed sand, silt, and clay, overlying granular strata, which overlie bedrock. The water table is located at El. 550 ft, or 8 to 20 ft below the surface.

To define the limits of contamination, a soil gas survey was performed using a hand-held hammer drill to drive probes 5 ft into the ground. Probing traced contamination to an off-site agricultural field and defined the general direction of contaminant movement (Figure 1). As the survey expanded, indications of volatiles abruptly stopped. It was suspected that a thicker clay unit was masking volatiles that might be present in the groundwater. The limits of contamination still had to be determined, but additional exploration was subject to the following constraints:

- Contamination was determined to be moving off-site, where stratigraphy was uncertain.
- The off-site property owner did not want drilled boreholes and monitor wells in his field.
- Poor weather conditions prevailed during the late winter months of 1990.
- Agency deadlines and budgetary limitations influenced the scope and schedule.

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**FIGURE 1 - EXTENT OF CONTAMINATION
BASED ON SOIL GAS SURVEY, APRIL 1988**

The needs of the off-site property owner had to be met while still allowing exploration to proceed. His concerns were addressed by negotiating permission to set a maximum of nine wells to monitor the extent of contamination, with assurances of minimal surficial disturbance to future crop cultivation.

A possible solution to the constraints of the agreement was to use a drilling rig to split-spoon sample soils, and to sample groundwater by using either a Hydropunch sampler or wellpoints. The exploration boreholes would be drilled and plugged until the extent of contamination had been determined. At this point, the monitor wells would be set. However, the cost of drilling and plugging numerous exploratory boreholes, and the overall ineffectiveness of drilling rigs to advance the Hydropunch in dense sands and gravels, made the use of a drill rig less than optimal.

Another solution was to use a cone penetrometer rig to explore subsurface conditions followed by conventional drilling to set monitor wells. This plan had the following advantages:

- The penetrometer rig has an enclosed work area so that bad weather does not significantly impact productivity.
- It is capable of collecting high quality, continuous stratigraphic information in real-time, both allowing for efficient penetrometer groundwater sampling and allowing a drill rig to eventually set monitor wells without additional physical sampling.
- It can be used to hydraulically push groundwater samplers into sands and fine gravels, under suitable conditions.
- Penetrometer operations do not generate possibly contaminated drill cuttings or fluids that require expensive disposal.
- The method does not result in large diameter holes that require extensive grouting to seal.
- Penetrometer operations are more rapid and thus less expensive than drilling operations.

Use of the penetrometer exploration/drill rig well installation approach was decided upon as the most cost- and performance-effective solution for this project. An analytical laboratory was set up on-site allowing for successive exploration points to be optimally chosen on the basis of contaminant plume detection, rather than on an arbitrary grid pattern.

PENETROMETER TECHNIQUE AND EQUIPMENT

Penetrometer methods are being used with increasing frequency, and they provide significant advantages for geo-environmental exploration. The technique uses a large hydraulic ram to push small (1.5 to 2.5 inch) diameter probes into the ground without drilling a borehole (Figure 2). Instrumented probes, called penetrometers, provide semi-direct and direct information on geotechnical, hydrogeological, and geochemical site conditions. Penetrometer samplers are used to obtain physical samples of subsurface materials.

Penetrometer methods are used to their greatest advantage in sand, silt and clay deposits. Penetrometer profiling (Figure 3) is continuous and accurate, and measurements (sounding

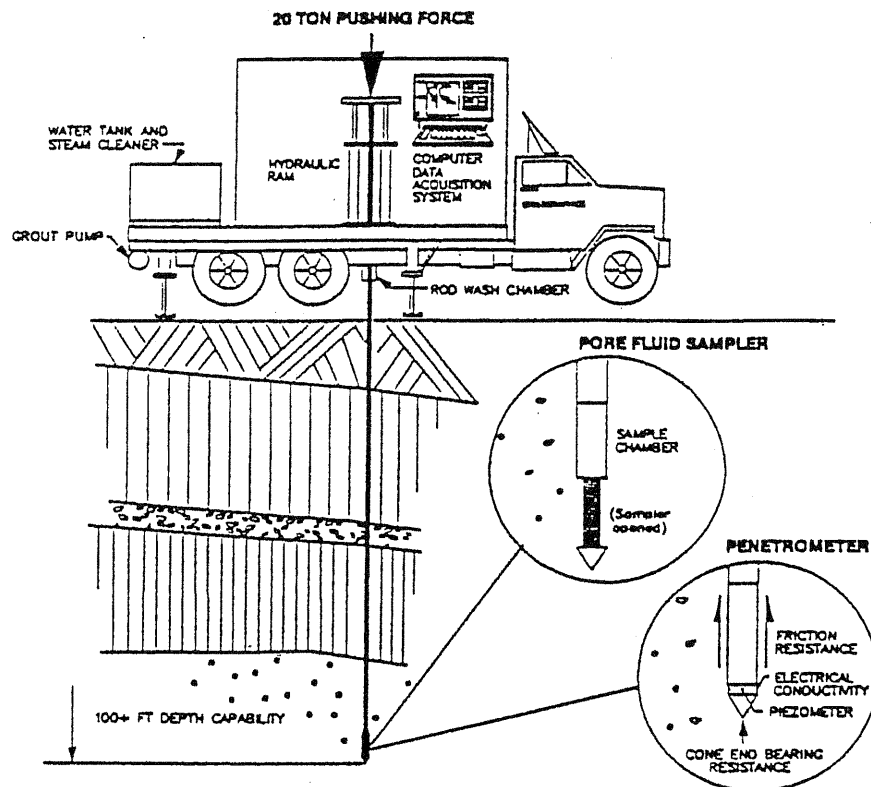


FIGURE 2 - PENETROMETER SUBSURFACE EXPLORATION SYSTEM

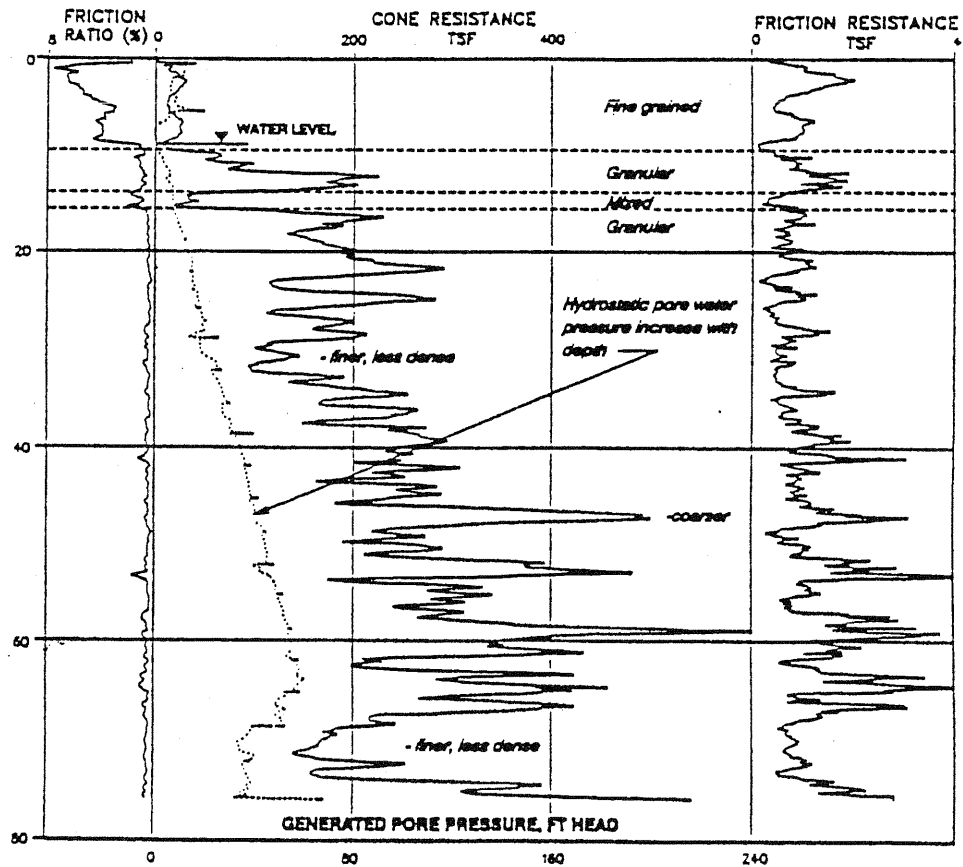


FIGURE 3 - CPTU SOUNDING LOG C-7

logs) are reliably interpreted for definition of aquifers and confining layers. Lateral continuity of layers is readily apparent from a series of adjacent penetrometer soundings.

Site disturbance and waste material disposal is minimized because no cuttings or drilling fluids are generated during penetrometer operations. Downhole equipment is steam cleaned during retrieval. Personnel exposure to contaminants is much less than exposures during drilling. Heaving sands pose little problem, as no open hole actually exists during penetrometer advance.

Three hundred to 900 ft of geotechnical soundings can be performed in a day, depending on access and project requirements. The special demands of geo-environmental investigations decrease daily footage, but productivity is still significantly higher than that using drilling rigs. Sounding depths in excess of 100 ft can be achieved, depending on site stratigraphy.

The penetrometer is mounted at the end of a series of heavy-wall sounding rods. A hydraulic ram is used to push the penetrometer into the ground at a rate of 4 ft per minute. Electronic signals from downhole sensors are transmitted by a cable, strung through the sounding rods, to a computer inside the penetrometer rig. Data are recorded at depth intervals of 3/8 to 3/4 inch. A real-time data display is monitored for evaluation of test performance and for immediate definition of site conditions. At the end of a sounding, the penetrometer and sounding rods are retrieved and decontaminated. Open hole can be grouted where cross-contamination between layers may occur. Open hole was allowed to collapse at the end of each sounding for this project, due to the lack of confining layers at the site.

A special truck is used to house, transport, and deploy the penetrometer. Twenty tons of truck weight and ballast are used to counteract the thrust of the hydraulic ram. The work area is enclosed and includes heating and air conditioning. Computers, penetrometers, samplers, electrical power, lighting, compressed air, grout and water pumps, steam cleaner for equipment decontamination, 275 gal water tank, tools and spare parts are all included within the penetrometer truck, providing for self-contained operations.

Instrumentation

The basic electronic penetrometer consists of two separate soil shear resistance sensors, and is used to acquire information on soil strength and stratigraphy. Tests conducted with this penetrometer are called Cone Penetration Tests (CPT) and are specified by ASTM Standard D3441. Electronic CPT has been used for geotechnical engineering projects for more than 25 years, while less sophisticated, uninstrumented versions of the test (Dutch cone test) have been used since the 1930s.

Two laboratory grade, strain gage loadcells, mounted inside the penetrometer, are used to measure the soil shear resistance to penetration acting on the conical tip and along the cylindrical sides of the CPT penetrometer. CPT measurements are continuous, accurate and repeatable. The tip or cone end bearing resistance can respond to soil seams as thin as 2 to 4 inches. The side or friction resistance measurement has a resolution of about 6 inches.

A pressure transducer is added to the CPT penetrometer to additionally measure the soil pore water pressure response to penetration; this is called a Piezometric Cone Penetration Test (CPTU). CPTU piezometric data allow for the evaluation of soil saturation, water tables, potentiometric surfaces, and soil horizontal permeability in both aquifers and aquitards. The CPTU piezometric measurement has a resolution of about 1 inch.

Another penetrometer has been used to measure the shear resistance, piezometric response, electrical conductivity and temperature of penetrated soils. This penetrometer is useful in

detecting free hydrocarbon product on groundwater. This test type is termed CPTU-EC. Additional details on penetrometer instrumentation can be found in Strutynsky, et al., 1985 and 1991.

Groundwater Samplers

The Hydropunch and BAT penetrometer samplers were used to sample groundwater for this project. The Hydropunch sampler (Figure 4) consists of a stainless steel, shielded wellpoint and sample barrel assembly (Edge and Cordry, 1989); it is deployed using the heavy-wall penetrometer sounding rod. The shield prevents contamination of the sampler while pushing. When the shield is retracted, groundwater flows under in situ pressure conditions into the 500 ml sample barrel.

A water level indicator can be lowered to the top of the Hydropunch in order to determine sampler filling. The sample is isolated in the barrel by two ball check valves. The entire sampler is retrieved to the surface to pour off the sample and for sampler decontamination. The tip of the Hydropunch sampler must be at least 4 ft below the water table to acquire a sample.

The BAT sampling system (Figure 5) consists of a wellpoint that is internally sealed with a septum (Torstensson, 1984). After pushing the wellpoint to depth, an evacuated 35 or 70 ml vial, also sealed with a septum, is wirelined down the casing. A double-ended hypodermic needle, mounted in an adapter below the vial, pierces both the wellpoint and the sample vial septa, and allows fluids to flow into the vial. The septa seal as the sample vial is retrieved to the surface, maintaining the sample at in situ pressure conditions. This procedure may be repeated to develop the wellpoint and to obtain increased sample volumes.

The BAT Enviroprobe and Mk. 2 wellpoints were used during this study. The Enviroprobe is shielded, while the Mk. 2 tip is unshielded. The Mk. 2 filter is exposed to possible cross-contamination during deployment. A thin-wall AQ drill rod is used to deploy the BAT samplers, as the heavy-wall penetrometer sounding rods are too small to pass the BAT sample vial.

PENETROMETER DATA INTERPRETATION

Correlations between penetrometer data and soil type have been developed from observational criteria on adjacent CPT soundings and drilled and sampled boreholes (Douglas and Olsen, 1981). CPT soil classifications reflect the shear response of a soil to penetration. Soil shear response is not entirely controlled by grain size distribution. However, soil types evaluated from CPT data generally agree with those classifications based on soil grain size distribution methods, such as the Unified Soil Classification System (USCS).

The CPT cone end bearing resistance increases exponentially with increases in grain size. The cone end bearing resistance in dense sands ranges from about 150 to 300 tons per square foot (TSF), while the cone end bearing resistance in a stiff clay ranges from about 7 to 15 TSF. The proportion of CPT friction to cone end bearing resistance, termed friction ratio, is related to the fines content of a soil. The friction ratio is low in sands and high in clays. CPT measurements are computer analyzed using CPT soil classification charts (Figure 6) to quickly define site stratigraphy.

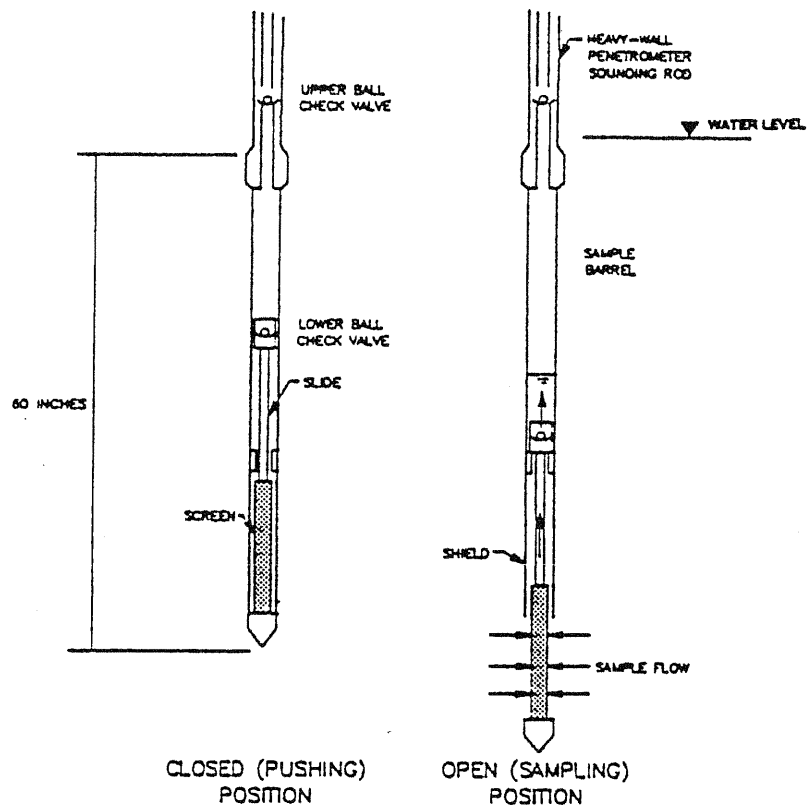


FIGURE 4 - HYDROPUNCH GROUNDWATER SAMPLER

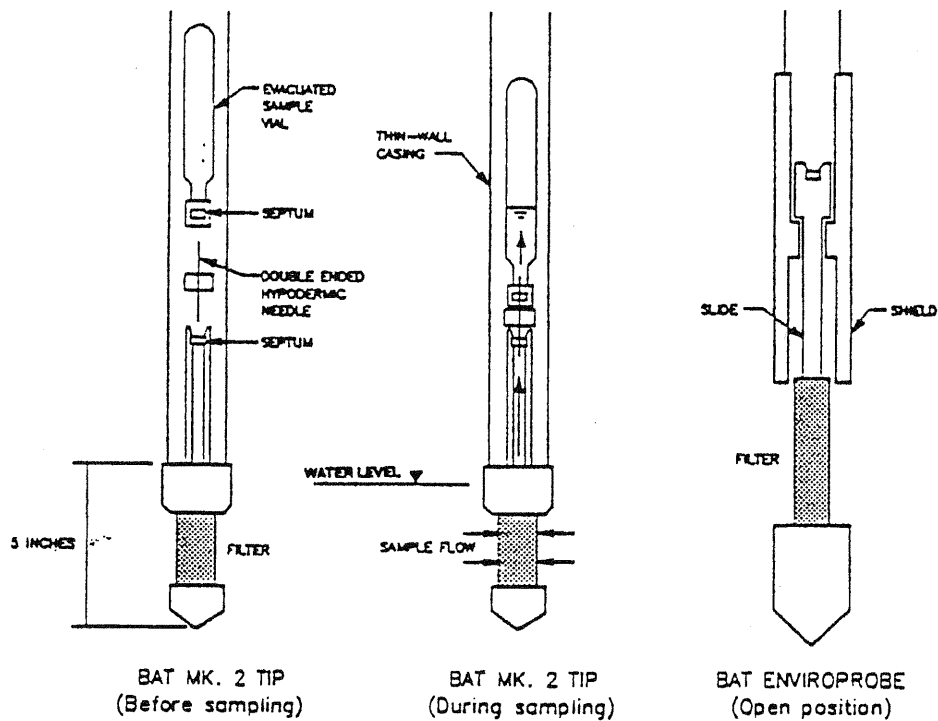


FIGURE 5 - BAT GROUNDWATER SAMPLING SYSTEM

Note: All dimensions approximate

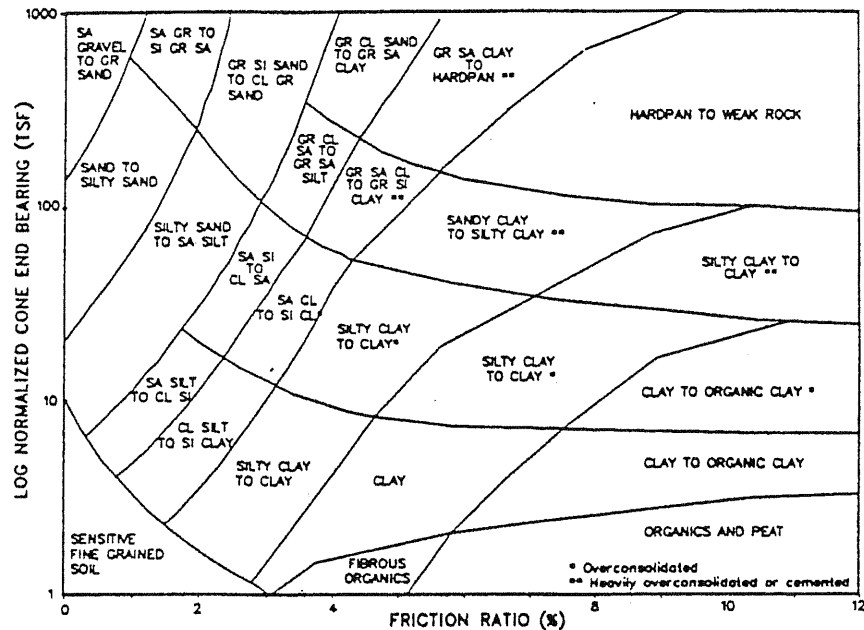


FIGURE 6 - CORRELATION CHART FOR CPT SOIL TYPES

The penetration of a saturated soil generates a localized pore water pressure field, in excess of equilibrium, around the penetrating probe. This generated pressure field dissipates quickly in soils of high permeability, so only the equilibrium pore water pressure field is measured in clean sands and gravels during Piezometric Cone Penetration Testing (CPTU). In low permeability soils, excess pore pressures require a significant amount of time to dissipate (Saines, et al., 1989). The dissipation of excess pore water pressure can be recorded as a function of time by pausing in the penetration process; this is termed a CPTU dissipation test. If the pauses are sufficiently long for all excess pressures to dissipate, measurements can be obtained of equilibrium potentiometric surfaces at multiple depths.

A CPTU dissipation test is somewhat similar to a falling head slug test, and can be used to calculate a value of soil horizontal permeability. However, tens to hundreds of feet of excess pore water pressure are induced in low permeability soils by penetrometer advance, as compared to several feet of head induced in a well during slug testing. The greater pressure changes during CPTU dissipation testing require soil compressibility effects to be included in analyses. The CPT cone end bearing resistance provides an index of compressibility for permeability computations.

ON-SITE LABORATORY

A Hewlett-Packard HP 5890A gas chromatograph (GC) with two electron capture detectors (ECD) was selected as the on-site analytical instrument for this project. It has the advantage of:

- Achieving laboratory results in the parts per billion (ppb) range
- Having a programmable oven which provides better separation of the eluting compounds for more accurate identification
- Qualifying and quantifying compounds as compared to internal reference standards

For the analysis of volatile organics, U.S. EPA SW-846 Method 3810 Headspace was used. Calibration procedures were performed using standard mixtures to establish the internal standard curve. Sample analysis consisted of pouring collected water samples into a 1.0 ml syringe, and injecting this sample into a nitrogen purged 40 ml septum vial. The sample was then heated in a 70 deg C water bath for 15 minutes; 100 ml of headspace was drawn from the center of the septum vial and injected into the GC for analysis.

For quality control (QC), every 10th unknown was run, per a modified U.S. EPA Method 3810. Modifications are to allow samples to equilibrate to 70 deg C, to use 40 ml vials with Teflon face septa, and to inject 100 ml of headspace gas. For each 10th unknown, four samples were injected to determine the quality of data. These consisted of a:

- Sample containing reagent water that had been carried through all stages of sample collection of unknown constituents
- Sample containing reagent water spiked with known amount of target analytes
- Sample containing unknown constituents
- Sample containing unknown constituents spiked with known amount of target analytes

In addition to the field analyses, select duplicate samples were sent to an off-site analytical laboratory for verification. Sample preparation, standards and QC were assisted by an analytical subcontractor.

PROGRAM RESULTS

CPTU soundings were performed at nine locations (Figure 7) during the period of February 6 to 13, 1990, for a total of 699 ft of test. Only partial days were worked on February 6 and 9, and no field work was done on February 10 when bulldozer support for access to locations in the wet, recently plowed field was not available. Twenty seven groundwater sampling attempts, with 22 successfully recovered samples, were also performed during this period. The sequence of penetrometer operations was as follows:

- 1) The penetrometer rig was set up, and a CPTU sounding was performed to determine hydrostratigraphy. The field sounding log was analyzed immediately to determine groundwater sampling depths.
- 2) The penetrometer rig was moved to provide about 5 ft of offset between the sounding and sampling hole to avoid vertical cross-contamination of samples.
- 3) Where sands were especially dense, or contained a high gravel content, a prepunch tool was pushed to nearly the sampling depth in order to facilitate sampler deployment.
- 4) The sampler was pushed to depth. Three groundwater samples were typically taken at each location at successively deeper penetrations (Figure 8), from shallow (15-31 ft), to intermediate (37-47 ft), to deep (68-90 ft). Three Hydropunch samplers were available on site, so as a sampler was being deployed, the others were being decontaminated.

The CPTU soundings revealed site soil conditions to have general lateral continuity, as can be seen on a stratigraphic cross-section (Figure 9). Site stratigraphy is summarized in Table 1.

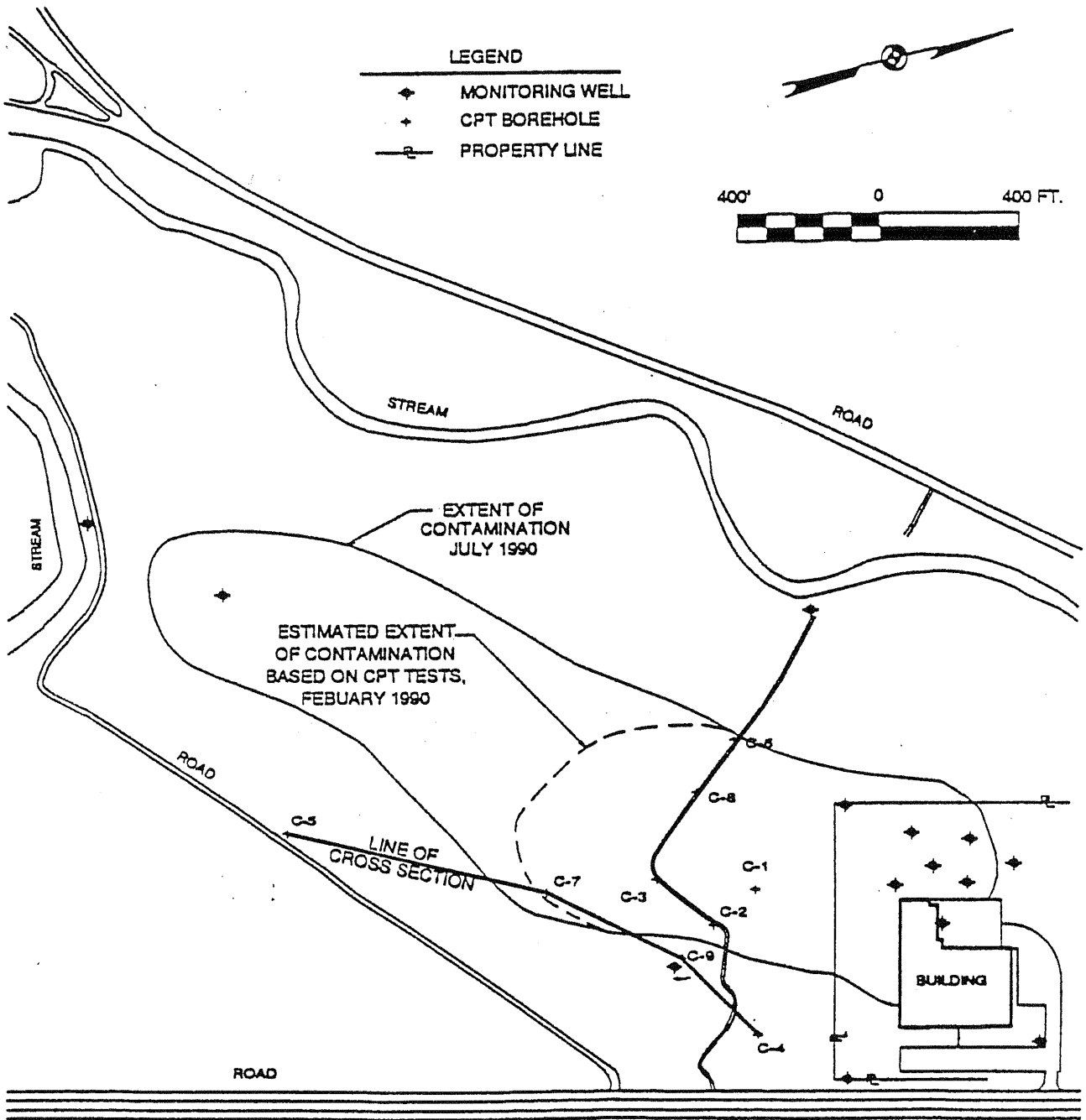


FIGURE 7 - LOCATION OF CPT TESTS, MONITOR WELLS,
LINE OF CROSS SECTION AND EXTENT OF TCE CONTAMINATION
AT THE TOP OF GROUNDWATER

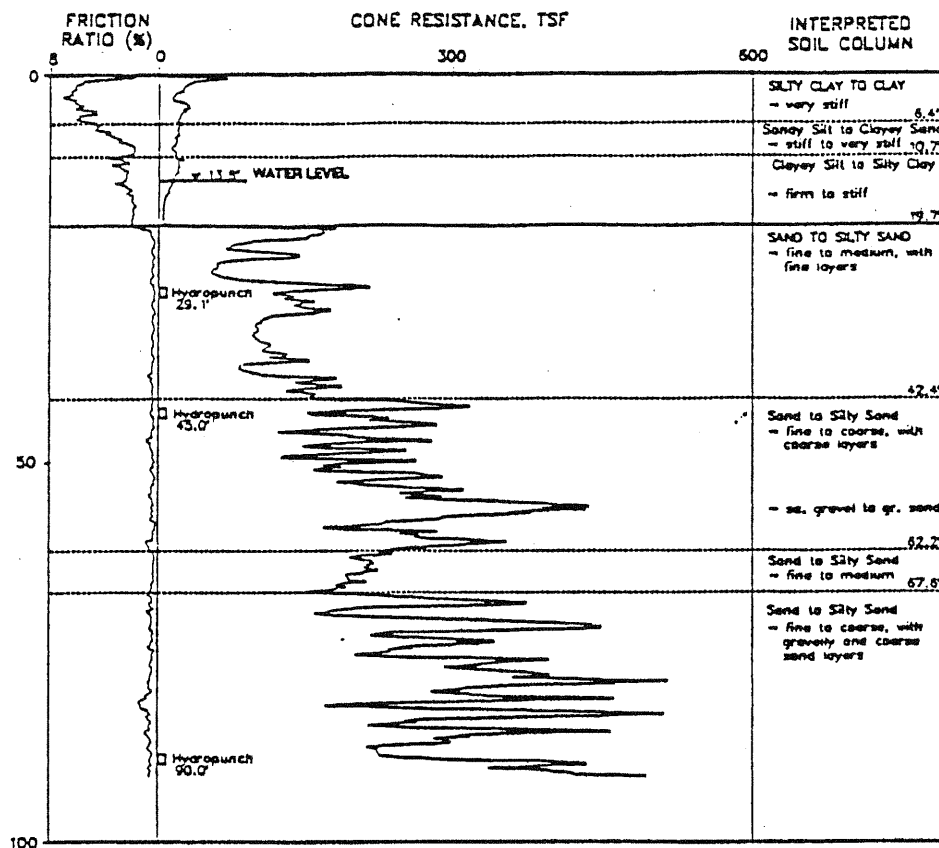


FIGURE 8 - INTERPRETED CPT SOUNDING LOG C-3 WITH SAMPLING DEPTHS

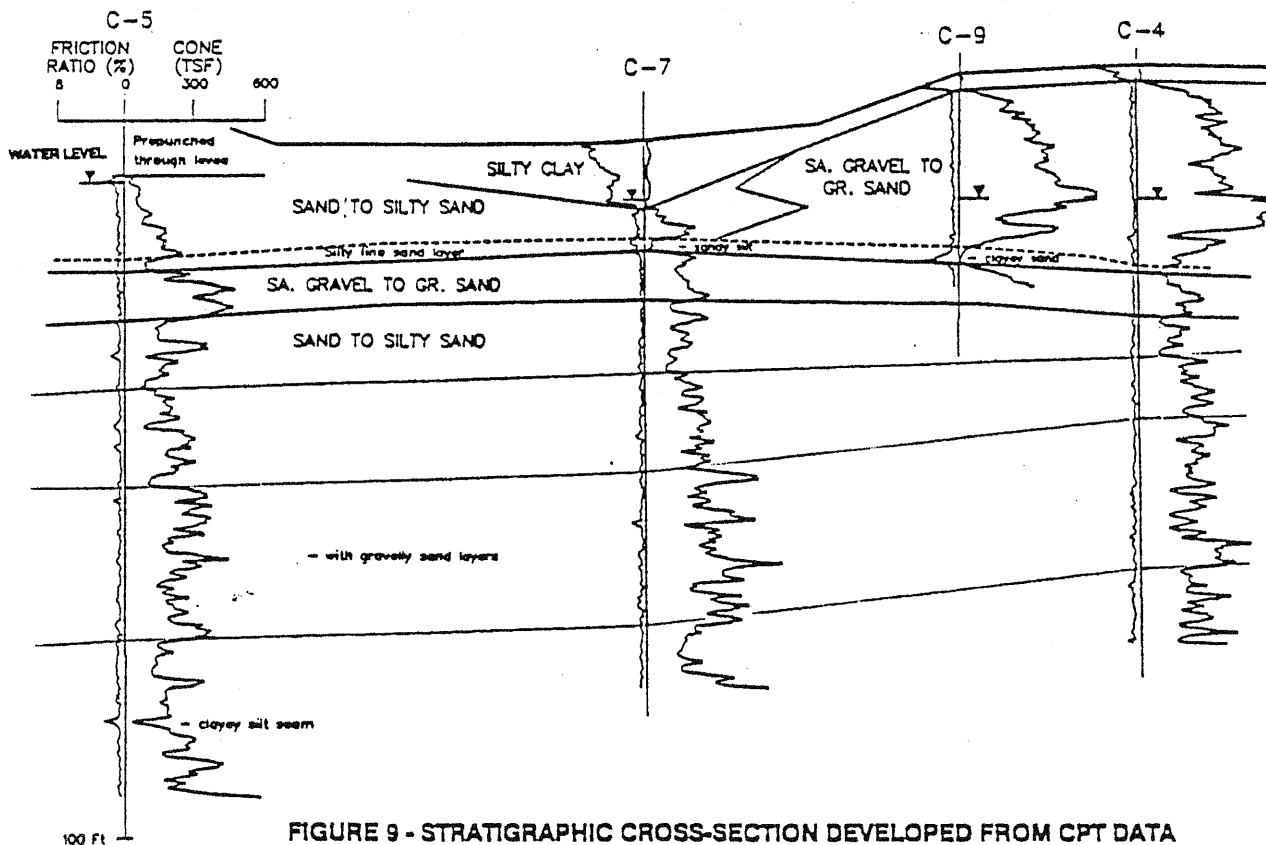


FIGURE 9 - STRATIGRAPHIC CROSS-SECTION DEVELOPED FROM CPT DATA

TABLE 1
CPTU SOIL STRATIGRAPHY

STRATUM NUMBER	DEPTH	DESCRIPTION	INCLUSIONS
1	0 ft to 2-3 ft (0 ft to 9-20 ft in C-3, C-7 & C-8)	Very stiff to firm sandy clay to clayey silt, with increasing silt content with depth	
2	2-3 ft to 75-94 ft (completion depth)	Medium dense to dense sand to silty sand, with layers of gravelly sand and sandy gravel.	a) a seam of cl. silt at 84 ft at C-5 b) sa. silt from 13.8 to 15.5 ft at C-7 c) cl. sand from 24.4 to 26.6 ft at C-9

A highly detailed characterization of the site geology was obtained from the CPTU sounding logs. Split-spoon sampling during drilled investigations often results in poor recovery in sands and gravels, which make detailed description impossible. In contrast, the CPTU data provided a continuous record of the material being penetrated, thus allowing even minor changes to be recorded accurately (Figures 3, 8 and 9). The high resolution of the CPTU data was important in finding zones of higher permeability for future groundwater remediation and also verified the lack of confining layers at the site. The assumption of a thick, surficial clay zone interfering with the soil gas survey was verified by soundings C-3, C-7 and C-8 (Figures 3, 8 and 9).

Groundwater conditions were measured using CPTU piezometric data (Figures 8 and 9). Water table measurements were also taken using a water level indicator lowered into the open hole left after a sounding. The potentiometric surface at one groundwater sampling depth was measured by allowing the sounding rod string to fill through the Hydropunch sampler.

The Hydropunch sampler was successfully used in obtaining 22 groundwater samples out of 27 attempts. In general, the Hydropunch sampler worked well, especially in regards to depth capacity. Some disadvantages of the Hydropunch sampler were:

- Lack of definitive feedback as to whether the shield opened
- Minor galling and seizing of sampler parts as is common with unlubricated stainless steel assemblies
- Slight bending of the sample barrel during pushing with forces in excess of 10 tons

The BAT groundwater samplers were used during two sampling attempts at the site. The thin-wall deployment casing buckled while pushing the Mk. 2 wellpoint through the shallow gravelly sands at location C-2, which precluded taking a sample. The Enviroprobe was successfully used at location C-5 and illustrated some of the features of the BAT system, including the retrieval of multiple samples and rapid feedback as to wellpoint shield opening.

Groundwater samples were analyzed at the on-site laboratory for four organic volatiles that had been identified in the groundwater. TCE was found to be the most prevalent contaminant. Several samples were sent to an off-site laboratory for verification of the field results. On-site analytical results were consistently lower than results generated in the off-site laboratory. However, the on-site laboratory results were sufficiently accurate to guide the location of successive exploration locations.

CONCLUSIONS

The approach of using a truck-mounted penetrometer rig, CPTU soundings, penetrometer groundwater samplers and an on-site GC laboratory to conduct geo-environmental site characterization studies, in combination with drill rigs to set monitor wells, proved to be highly advantageous in terms of:

- Minimal site disturbance and generation of wastes
- Collection and analysis of high quality, high resolution hydrostratigraphic data, with little or no downtime
- Optimal positioning of drill rig installed monitor wells, with little additional split-spoon sampling or geotechnical laboratory testing
- Meeting deadlines and budgetary constraints

Based on the results of the penetrometer investigation, monitor wells were installed using drill rigs along the downstream perimeter of the contaminant plume. Little split-spoon sampling was performed during well installation as stratigraphy had already been defined to a high degree by the CPTU sounding data. Several rounds of monitor well sampling have verified the accuracy of penetrometer groundwater sampling and the use of field analytical testing.

REFERENCES

- Edge, R.W., K. Cordry, 1989. The Hydropunch: An In Situ Sampling Tool for Collecting Ground Water from Unconsolidated Sediments. Ground Water Monitoring Review, Vol. IX (3), pp 177-183.
- Douglas, B.J., R.S. Olsen, 1981. Soil Classification Using Electric Cone Penetrometer. Cone Penetration Testing and Experience, ASCE, pp 209-227.
- Saines, M., A.I. Strutynsky and G. Lytwynyshyn, 1989. Use of Piezometric Cone Penetration Testing In Hydrogeologic Investigations. Presented at the First USA/USSR Hydrogeology Conference, Moscow, USSR.
- Strutynsky, A.I., B.J. Douglas, L.J. Mahar, G.F. Edmonds, and E. Hency, 1985. Arctic Penetration Test Systems. Civil Engineering in the Arctic Offshore, ASCE, pp 162-168.

Strutynsky, A.I., R. Sandoford, and D. Cavaliere, 1991. Use of Piezometric Cone Penetration Testing with Electrical Conductivity Measurements (CPTU-EC) for Detection of Hydrocarbon Contamination in Saturated Granular Soils. Accepted for publication, Ground Water and Vadose Zone Investigations, ASTM.

Torstensson, B-A, 1984. A New System for Groundwater Monitoring. Ground Water Monitoring Review, Vol. IV (4), pp 131-138.

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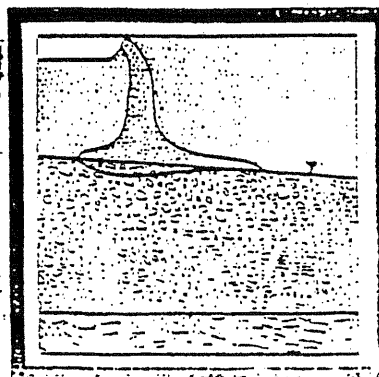
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USE OF PIEZOMETRIC CONE PENETRATION TESTING
IN HYDROGEOLOGIC INVESTIGATIONS

By M. Saines, A. Strutynsky, and G. Lytwynyshyn

Presented at the

First USA/USSR Hydrogeology Conference
Moscow, USSR
July, 1989

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USE OF PIEZOMETRIC CONE PENETRATION TESTING
IN HYDROGEOLOGIC INVESTIGATIONS

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ABSTRACT

Piezometric Cone Penetration Testing (CPTU) is a powerful exploration tool for hydrogeologic investigations. CPTU consists of pushing a 1.4 to 1.7-inch diameter (3.6 - 4.4 cm) penetrometer to depths of 150 feet (45 m) or more in unconsolidated deposits of clay, silt, and sand. Sensors mounted inside the penetrometer provide data for the instantaneous evaluation of the following hydrogeologic parameters:

- 1) stratigraphy and lithology - identification of aquifers, aquitards, and definition of the lateral continuity of these units;
- 2) the position of the water table/potentiometric surface in sands and the water table-capillary fringe in silts and clays;
- 3) the hydraulic head in confined aquifers;
- 4) the slope of the water table or potentiometric surface, and therefore the direction of groundwater movement;

- 5) the vertical gradient by determining the head at various depths in each CPTU sounding; and
- 6) the permeability and transmissivity of aquifers.

The CPTU penetrometer is hydraulically pushed into the ground using a 20 ton (180 kN) truck. CPTU data are gathered by an on-board computer as continuous functions of both depth and time. Data consist of cone end bearing resistance, friction sleeve resistance, penetrometer deviation from vertical, and pore water pressure response to penetration. Lithologies are obtained by computer processing of CPTU sounding data using observational classification criteria based on an extensive library of comparisons between CPTU and drilled and sampled boreholes. Small diameter standpipe piezometers may also be installed using 20 ton CPTU equipment at a significant savings of time and money compared to drilling methods.

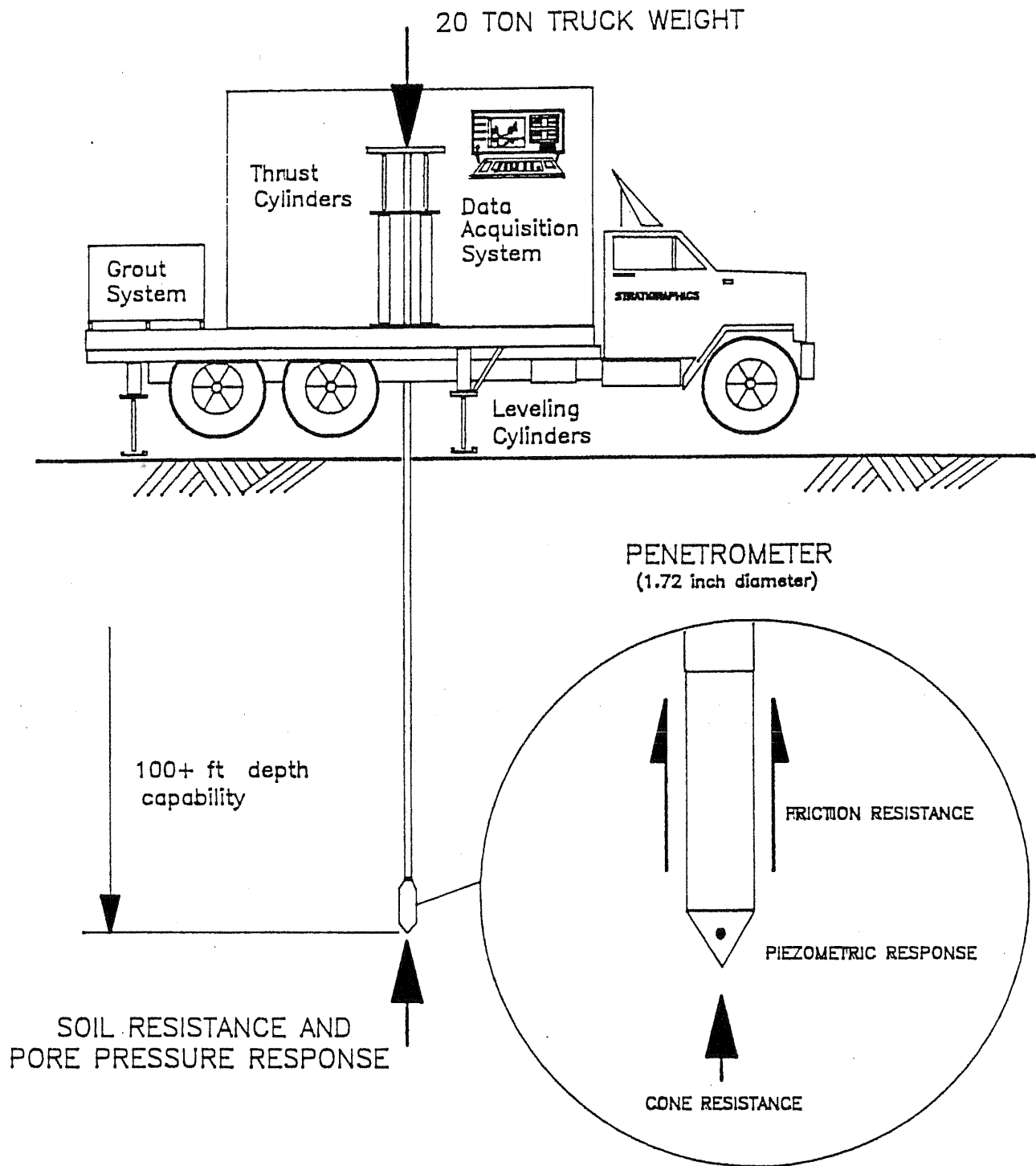
The continuous CPTU sounding yields accurate hydrogeologic data quickly and at low cost without drilling, sampling or laboratory testing. Test production rates vary from about 300 feet (90 m) to over 800 feet (250 m) per day, depending on project requirements. Cost comparisons are presented illustrating possible savings of up to 85 percent over continuously sampled and tested borings, and cost savings of 35-65 percent over the cost of borings with samples and laboratory tests at 5-foot intervals.

Limitations of the Piezometric Cone Penetration Test method include penetrometer refusal by coarse gravels, cobbles, and bedrock, or excessive friction on sounding rods during deep soundings. Depths of as much as 150 to 230 feet (45 to 70 m) can be reached with 20 ton CPTU equipment, depending on site stratigraphy.

INTRODUCTION

In unconsolidated deposits of sand, silt, and clay, Piezometric Cone Penetration Testing (CPTU) provides excellent hydrogeologic data accurately, quickly, and inexpensively. CPTU consists of smoothly pushing a small diameter instrumented probe (penetrometer) into the ground using a hydraulic ram (Figure 1). High technology sensors mounted inside the penetrometer provide data for the evaluation of soil type, soil strength, and pore water response of penetrated soils. Testing is rapid and precise. In environmental investigations there is lessened exposure of personnel to potentially contaminated soil and groundwater. Small diameter, standpipe piezometers can also be installed using CPTU equipment; these piezometers can be tested for permeability, sampled for water quality, and monitored for water level changes over time.

Traditional site characterization studies have involved performing numerous soil boring, soil sampling and piezometer and/or observation well installations. These studies can be expensive, time consuming and yield somewhat subjective results. For example:



PIEZOMETRIC CONE PENETRATION TEST STRATIGRAPHICS

FIGURE 1 — SCHEMATIC OF ELECTRONIC PIEZOCONE PENETRATION TESTING

- 1) traditional discontinuous soil sampling is expensive, while continuous sampling is very expensive;
- 2) sample reliability is often degraded by poor recovery or other drilling problems;
- 3) transporting, handling, cataloging, classifying, testing and storing soil samples is both time consuming and expensive;
- 4) drilling and sampling results in disturbed soil samples;
- 5) field soil classification and geological logging are often subjective;
- 6) hydrogeological characterization requires expensive drilled well installations and testing;
- 7) long lag times, days or weeks, are often required for drilled wells to reach equilibrium conditions in low permeability soils; and
- 8) large quantities of fluids and cuttings are brought to the surface during drilling operations; in contaminated areas these fluids and cuttings require expensive handling and disposal.

CPTU suffers from none of these disadvantages. CPTU used independently for initial studies or in conjunction with a limited boring program, results in a less costly but more detailed site characterization study. (Olsen and Farr, 1986.)

A major benefit of using CPTU soundings for stratigraphic correlation is that the data are objective, whereas a geologist's visual log is subjective and dependent upon the quality of recovered samples. CPTU sounding data can delineate very small features (+/- 1 inch or 2.54 cm) which can be missed if sample quality is poor in continuously sampled boreholes; much thicker units, of course, are regularly missed in discontinuously sampled boreholes.

CPTU soundings are also more reliable than geophysical logging. CPTU response is directly related to primary soil characteristics such as grain size, void ratio, and permeability. Geophysical logs, such as natural gamma and electrical logs, reflect secondary soil properties such as radioactivity and electrical conductivity. Geophysical logging may also be affected by borehole characteristics and geometry, and groundwater chemistry.

PRINCIPLES OF PIEZOMETRIC CONE PENETRATION TESTING

Data Acquisition

Test Equipment and Procedures. CPTU consists of pushing an instrumented penetrometer into the ground while continuously recording the soil resistance and pore pressure response to penetration. A profile of the in situ soil mechanical properties is obtained rapidly and accurately. The penetrometer advance rate of 4 ft/min (2 cm/sec) is such that drained and undrained conditions exist while penetrating sands and clays, respectively. Both American ASTM and European ISSMFE standards specify various aspects of penetrometer test procedures.

The penetrometer is mounted at the end of a series of sounding rods. A set of hydraulic rams is used to push the penetrometer and rods into the soil at a constant rate. The thrust of the rams automatically varies according to soil resistance. A self-contained, 20 ton (180 kN) dead weight truck is used to counteract the thrust of the hydraulic rams, and to house and transport the test equipment (Figure 1). Test production rates vary from about 300 feet (90 m) to over 800 feet (250 m) per day, depending on terrain, pore pressure dissipation monitoring requirements, and hole grouting.

All work is performed from inside the vehicle, so testing can efficiently proceed in all types of weather. Additionally, the enclosed work space shields activities from onlookers, resulting in a much lower visual presence than that associated with drill rig operation.

The use of CPTU should not be planned for sites with shallow bedrock, or extensive boulder, cobble and coarse gravel deposits. Practical depths of penetration are typically limited by the reaction weight of the truck carrying the equipment. Twenty ton dead weight systems can be expected to have sufficient thrust to penetrate as deep as 150 to 230 feet (45 to 70 m) at many sites.

Penetrometer. The penetrometer soil mechanical load sensors consist of a conical tip and cylindrical sleeve. The conical tip has a 60 degree apex angle and a projected cross sectional area of 15 square centimeters; the cylindrical sleeve has a surface area of 200 square centimeters. Elements with other sizes are also in common use.

The interior of the penetrometer consists of two strain gauge load cells that allow simultaneous measurement of cone tip and sleeve loads during penetration. Continuous electrical signals from the downhole load cells are transmitted by an electrical cable strung through the hollow sounding rods to the field computer inside the CPTU truck. The technician monitors a real time display of subsurface soil resistance, pore pressure response, and penetrometer deviation from vertical, for evaluation of test performance.

Piezometer. A miniature diaphragm type pressure transducer is mounted inside the conical tip of the penetrometer. This transducer is coupled to the soil through a fluid filled porous filter, and is used to measure soil pore pressure response as a function of depth during penetration. The dissipation of excess pore pressures can be recorded as a function of time by stopping the penetration process at any particular depth. By allowing sufficient time to pass, a measurement can be obtained of the equilibrium hydraulic head at that depth.

Signal Conditioning and Recording. Data are digitally recorded using a 16 bit A/D (1 part in 32,728) data logger and field computer. Data is displayed in real time on the computer screen during testing, allowing for immediate evaluation of test performance. Preliminary hard copy is provided at the end of a test. Additional quality assurance procedures and report ready data presentation processing are performed in-house after completion of field work. Measurement accuracy and data processing are greatly enhanced using this high resolution digital system.

A schematic of the CPTU equipment is presented in Figure 1. Further details of instrumentation and other sensors that can be used in conjunction with CPTU are presented in Strutynsky and others (1985).

Data Reduction

Measured data consist of depth, time, cone end bearing and friction sleeve resistances, total load on penetrometer, pore pressure response, and penetrometer inclination. Data are recorded and plotted at a 1 Hz frequency, or at about 0.7-inch (2 cm) intervals. The following parameters are computed at each depth increment to enhance test interpretation:

Friction ratio, FR, in percent

$$FR = fs/qc \times 100 \quad (\text{Eq. 1})$$

Pore pressure ratio, Bq:

$$Bq = (u - u_h) / (qc - S_v) \quad (\text{Eq. 2})$$

where:

fs is the measured sleeve resistance, in TSF (tons per square foot);

qc is the measured cone resistance, in TSF;

u is the measured generated pore pressure response, in TSF;

uh is the measured or estimated equilibrium hydraulic head, in TSF;
and

Sv is the estimated total soil overburden pressure, in TSF.

Data Interpretation

Sounding Log. The continuous plot of CPTU data versus depth (sounding log) provides direct information on subsurface conditions. Layering is readily apparent, along with relative soil strength and consistency. Inspection of a series of continuous CPTU sounding logs helps to define site stratigraphy with greater ease and more accuracy than most borehole or geophysical techniques. Stratigraphic correlations are most easily made by comparing and overlaying consecutive CPTU soundings on a light table. Characteristic data signatures are visually matched, resulting in tracing of layer continuity across the site.

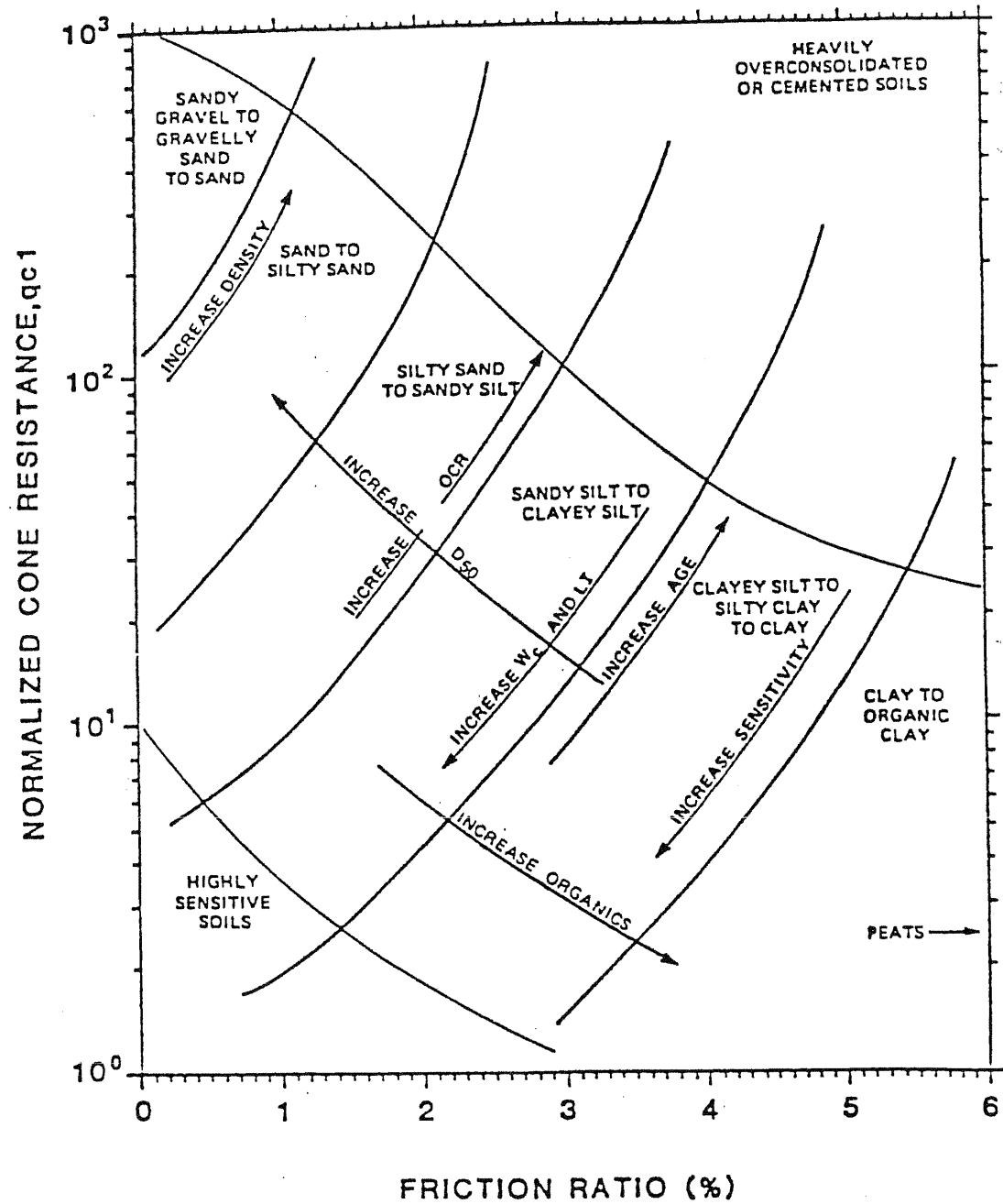
CPTU Soil Classification. CPTU classifications are based on at least 30 years of observational criteria from great numbers of side by side penetrometer soundings and drilled and sampled boreholes. In general, soils that exhibit high cone resistance and low friction ratio are sands while layers with low end bearing and high friction ratio are clays. Mixed soils, such as clayey sands and silts, exhibit intermediate trends. A detailed description of the use of CPT (no pore pressure measurement) data for soil classification is presented in Douglas and Olsen (1981). An example CPT classification chart is presented in Figure 2. An extension of these classification techniques to include results of CPTU (CPT with pore pressure measurements) is presented in Robertson and others (1986); an example CPTU classification chart is presented in Figure 3.

Cone End Bearing Resistance (q_c). A measurement of a soil's bearing capacity is provided by the output of the load cell connected to the conical tip of the penetrometer. Soil bearing capacity depends primarily on grain size, and on the effects of grain size on permeability and compressibility.

Bearing capacity increases exponentially with grain size. Clays have low bearing capacity, while silt bearing capacity is typically somewhat higher. Sands have very high bearing capacity - the bearing capacity of a sand is from one to two orders of magnitude greater than that of a clay. Thus, the cone end bearing resistance is extremely sensitive to sand content.

Typical cone end bearing measurements are:

- 1) 2 - 12 TSF (0.2 - 1.2 MN) in Holocene clays, depending on depth;
- 2) 20 - 40 TSF (2 - 4 MN) in desiccated clays, and in preloaded Pleistocene and older clays;
- 3) 5 - 50 TSF (0.5 - 5 MN) in clayey silts to sandy silts;
- 4) 30 - 60 TSF (3 - 5 MN) in loose fine sands;

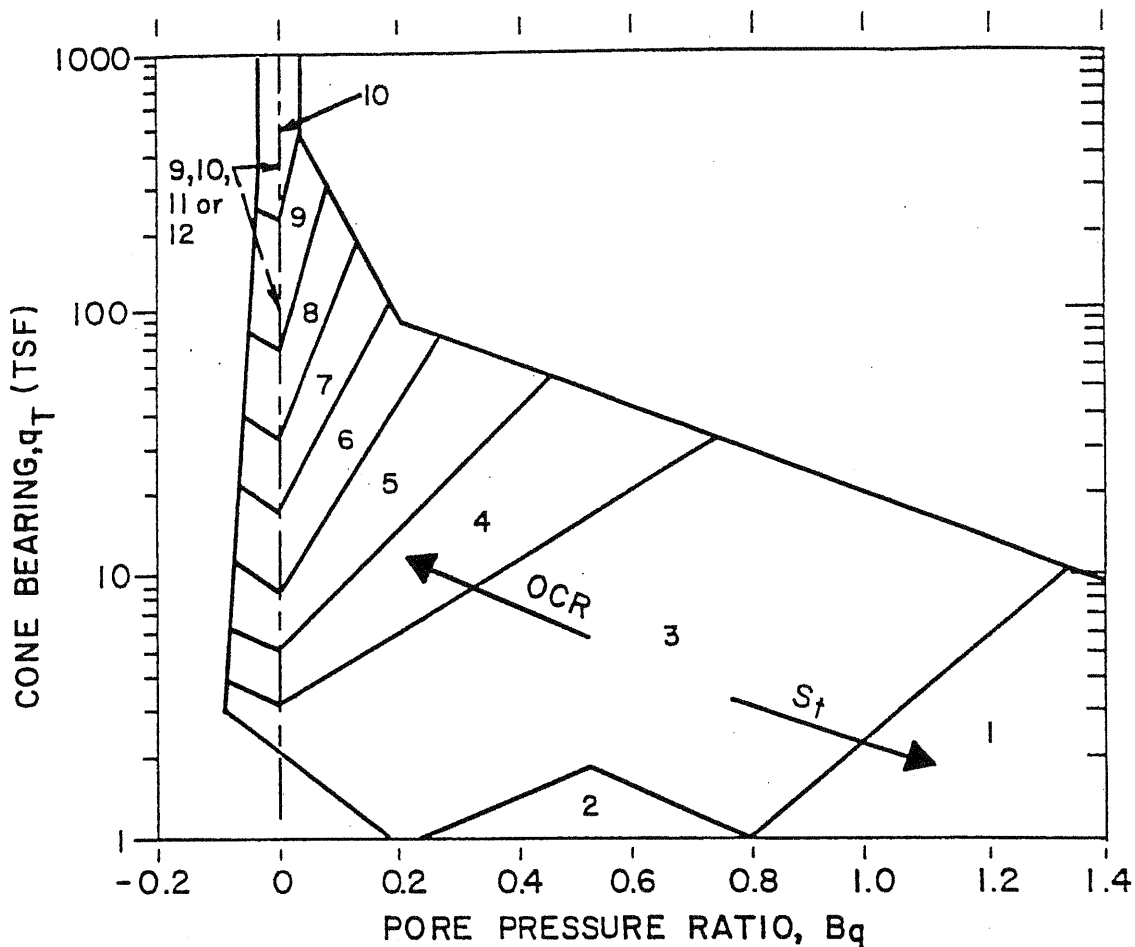


SOIL BEHAVIOR TYPE CLASSIFICATION CHART

After Douglas and Olsen (1981)

STRATIGRAPHICS

FIGURE 2 — CPT SOIL BEHAVIOR TYPE CLASSIFICATION CHART



ZONE	SOIL BEHAVIOUR TYPE
1	SENSITIVE FINE GRAINED
2	ORGANIC MATERIAL
3	CLAY
4	SILTY CLAY TO CLAY
5	CLAYEY SILT TO SILTY CLAY
6	SANDY SILT TO CLAYEY SILT
7	SILTY SAND TO SANDY SILT
8	SAND TO SILTY SAND
9	SAND
10	GRAVELLY SAND TO SAND
11	VERY STIFF FINE GRAINED (*)
12	SAND TO CLAYEY SAND (*)
(*)	OVERCONSOLIDATED OR CEMENTED

SOURCE: ROBERTSON & OTHERS (1986)

**FIGURE 3 — PROPOSED SOIL BEHAVIOR
TYPE CLASSIFICATION SYSTEM
FROM CPTU DATA**

- 5) 150 - 400 TSF (15 - 40 MN) in dense sands; and
- 6) 200 - 800 TSF (20 - 80 MN) in gravelly sands and gravels.

Friction Sleeve Resistance (f_s). The friction sleeve resistance of a soil varies approximately as a linear function of grain size and is inversely proportional to porosity. This measurement does not vary as dramatically as the cone end bearing resistance in stratified deposits. The friction sleeve resistance reflects soil large strain (disturbed) properties, as the sleeve interacts with soil that has already undergone bearing capacity failure, induced by the cone tip.

Friction Ratio (FR). The friction ratio is calculated by dividing the friction sleeve resistance by the cone end bearing resistance, expressed as a percentage (Equation 2). The friction ratio is low (0.5 - 2 percent) in sands due to the very high cone end bearing in sand. The friction ratio is high in clays (3 - 8 percent), and intermediate (1 - 5 percent) in mixed soils such as clayey silts and sandy clays.

Generated Pore Pressure (u). The soil water pressure response to penetrometer insertion depends on soil saturation, permeability and compressibility. The pore pressure response in unsaturated soils is zero. No pore pressures in excess of equilibrium are generated, or the dissipation of generated excess pressures occurs much more rapidly than the 1 second response time of the CPTU piezometer-data acquisition system in high or medium permeability (k greater than about 1.0×10^{-4} cm/sec) soils. Thus in saturated, permeable, clean sands the CPTU pore pressure response provides a direct measurement of the hydraulic head at that depth.

As soil permeability decreases below about 1.0×10^{-4} cm/sec, soil volumetric distortion due to penetrometer advance results in excess pore pressure generation. These generated excess pressures dissipate much more slowly than the response time of the piezometer system. Generated pore pressures in saturated low to very low permeability soils reflect both permeability and compressibility effects. High generated pore pressures are measured in both low compressibility dirty sands or high compressibility clays.

Pore Pressure Ratio (B_q). The pore pressure ratio is calculated as the generated pore pressure (in excess of equilibrium) divided by the cone end bearing resistance (Equation 2). This normalized parameter is useful in discrimination of soil compressibility and permeability effects. Due to the high cone end bearing in low compressibility, dirty sands, the pore pressure ratio typically ranges from about 0.05 to 0.20 in these soils. In saturated, high compressibility clays, the pore pressure ratio typically ranges from about 0.4 to over 1.0. Thus, the CPTU pore pressure ratio is useful, along with cone end bearing and friction ratio, in classifying lower permeability soils.

Pore Pressure Dissipations. By stopping the penetration process and monitoring the decay of generated pore pressure versus time, an in situ measurement is obtained of the equilibrium hydraulic head at that particular depth. The dissipation rate also reveals information on soil compressibility and permeability. Thus, another discrimination between dirty sands and clays can be based on the amount of time required to dissipate generated excess pore pressures. Long dissipation times are associated with clays, while short times indicate sands.

EXAMPLES OF THE USE OF PIEZOMETRIC CONE PENETRATION TESTING IN HYDROGEOLOGICAL INVESTIGATIONS

Stratigraphy

At a landfill in northeastern Illinois, CPTU soundings were used to investigate the hydrogeological conditions at proposed monitoring well locations. CPTU data were used to predetermine well design without expensive drilled continuous sampling. Stratigraphic continuity was quickly evaluated at the site by doing a series of CPTU soundings. The depths to the uppermost aquifer, extent of aquitards and locations of discontinuous sand lenses were readily identified on cross-sections developed from the CPTU sounding logs. Permanent monitoring wells were later constructed with hollow stem augers in the uppermost aquifer at the depths indicated by the CPTU soundings.

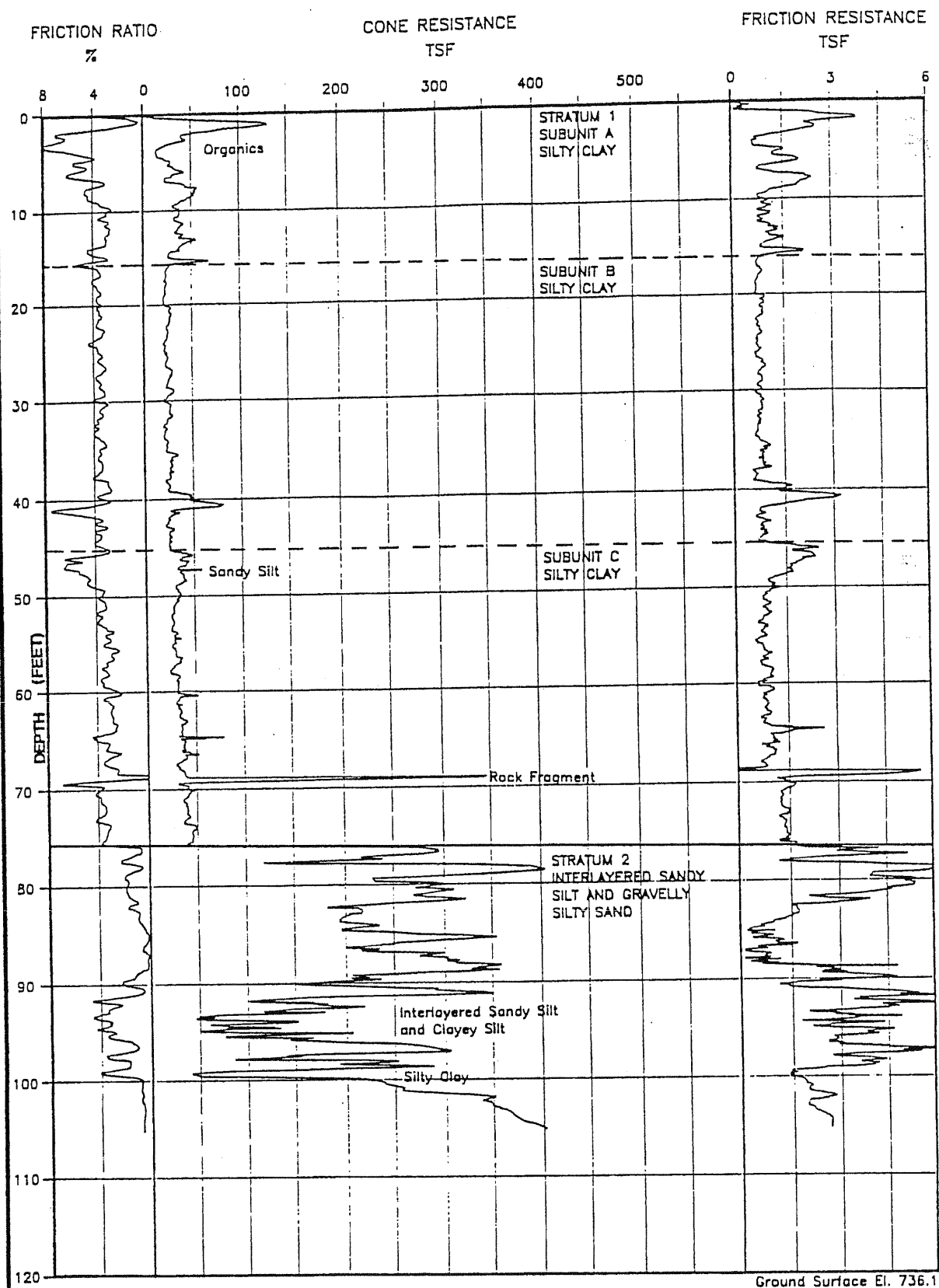
The CPTU soundings clearly defined the site stratigraphy - clay till overlying a sandy silt to silty sand aquifer. A plot of the cone resistance and friction ratio versus depth is shown in Figure 4 for CPTU Sounding D-6. Note the abrupt decrease in the friction ratio and large increase in the cone end bearing resistance at a depth of 76 feet indicating a sharp clay to sand transition. This sand unit was identified as the uppermost aquifer at the site.

The contrast between the clay till and sand is also very clear on the plot of the pore pressure ratio, B_q , versus cone end bearing resistance for the same sounding (Figure 5). The pore pressure ratio, B_q , is high in saturated clays and low in sands. Above 76 feet, the high pore pressure ratio and low cone end bearing resistance indicate a saturated clay, while the sand unit below 76 feet exhibits a low pore pressure ratio and high cone end bearing resistance.

Position of the Water Table or Capillary Fringe

CPTU soundings can be used to determine the position of the water table or zone of saturation. Figure 6 shows the water table in a thick sand unit as interpreted from the generated pore pressure plot of Sounding P-17 at JFK Airport, New York, New York. A CPTU sounding log in a clay till is presented in Figure 7. The position of the top of the fully saturated zone in the clay was about 7 feet below the ground surface at this location.

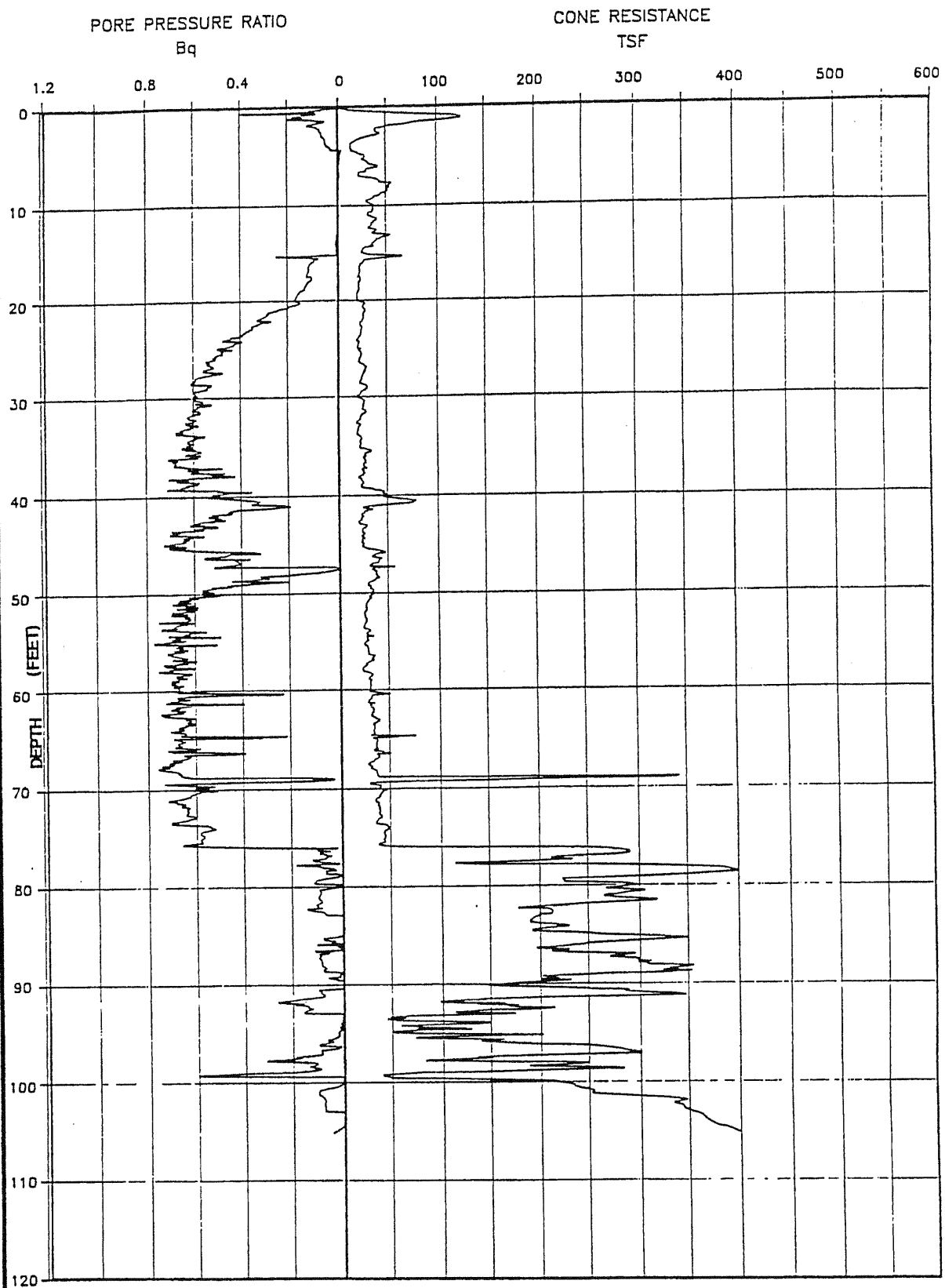
CONE PENETRATION TEST



STRATIGRAPHICS

FIGURE 4 — FRICTION RATIO VERSUS CONE RESISTANCE FOR CPTU SOUNDING D-6, WINTHROP HARBOR, ILLINOIS

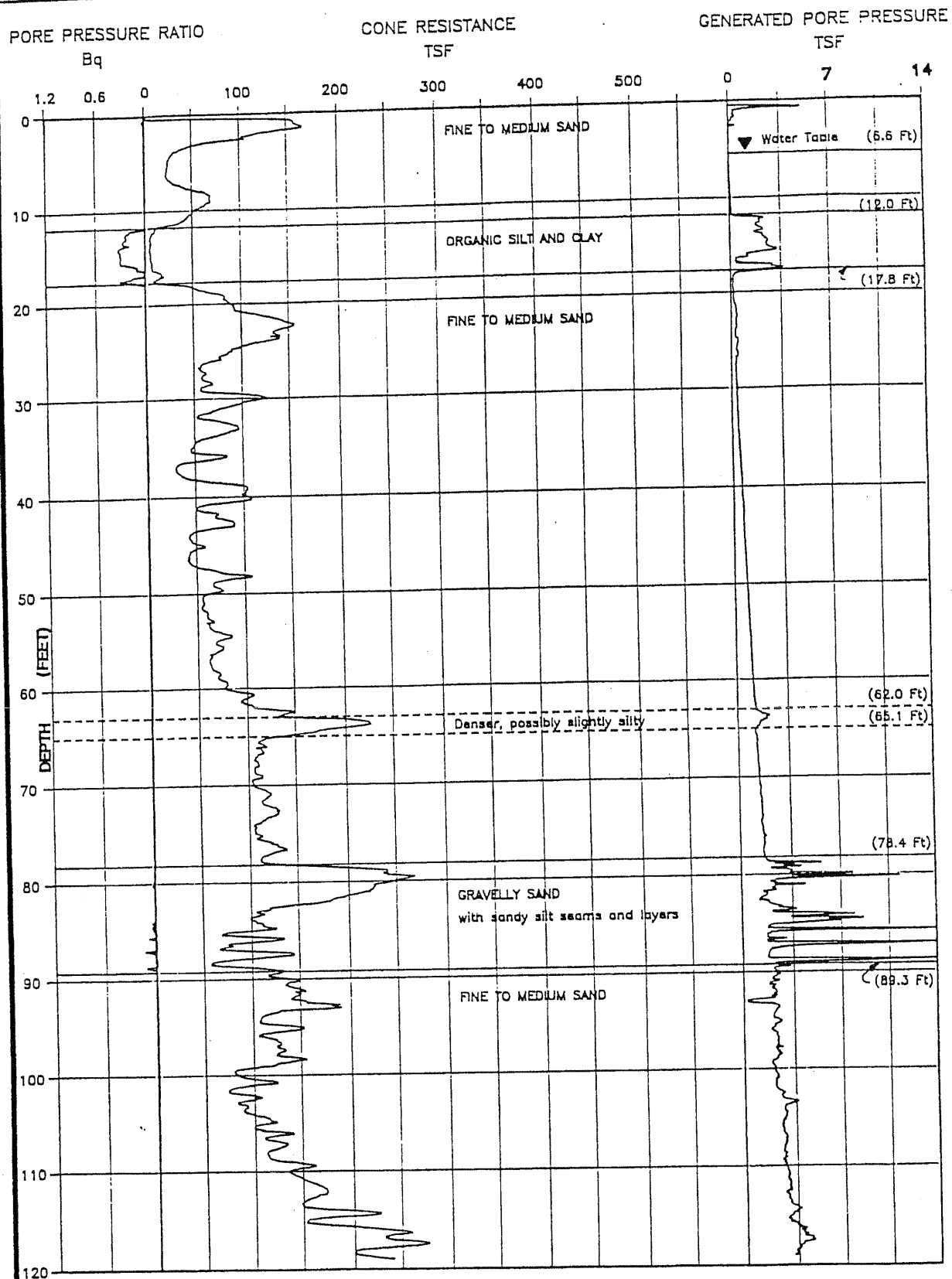
CONE PENETRATION TEST



STRATIGRAPHICS

FIGURE 5 - PORE PRESSURE RATIO VERSUS CONE RESISTANCE FOR CPTU SOUNDING D-6, WINTHROP HARBOR, ILLINOIS

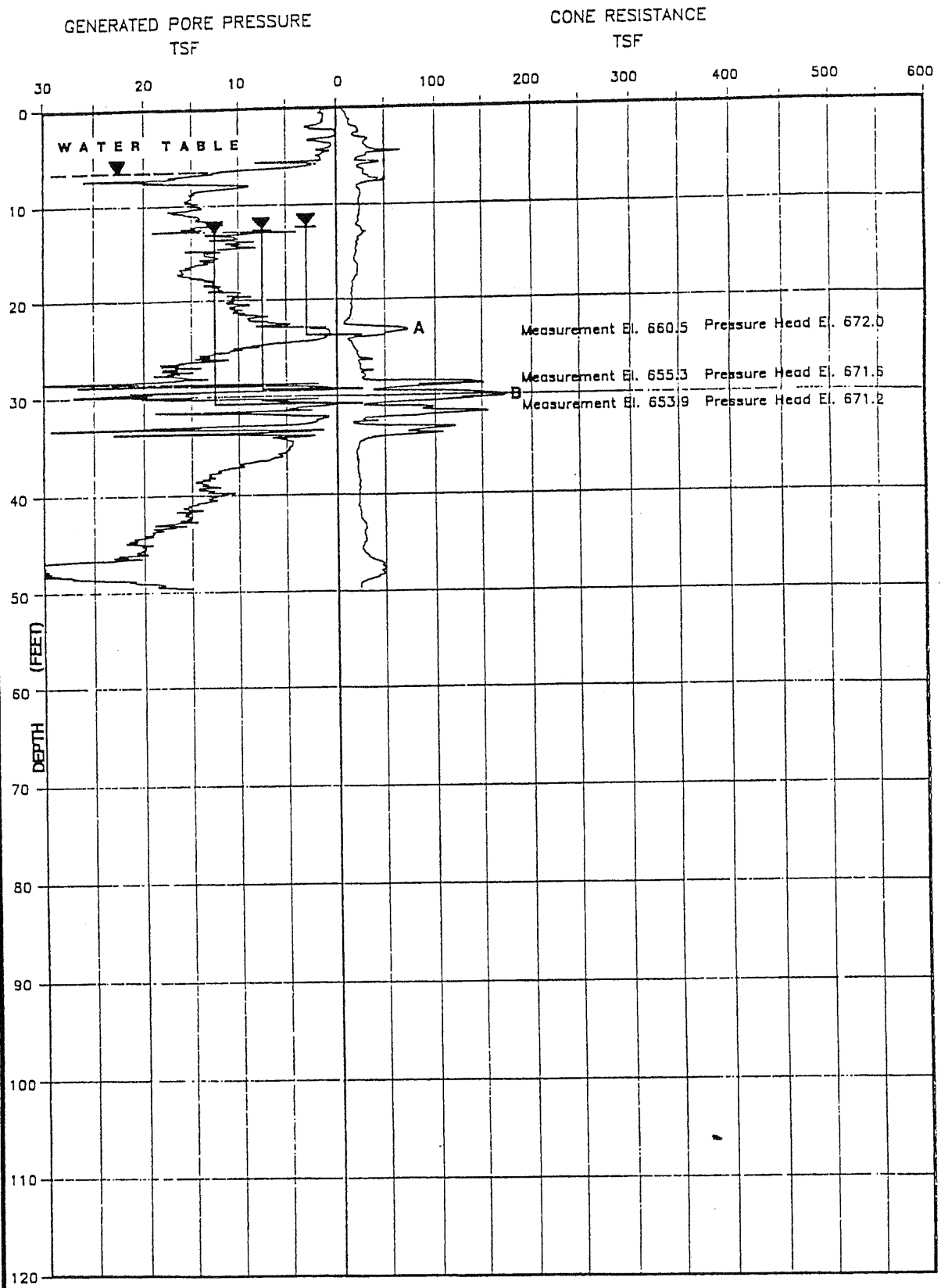
PIEZOMETRIC CONE PENETRATION TEST



STRATIGRAPHICS

FIGURE 6 - PORE PRESSURE RATIO VERSUS CONE RESISTANCE AND GENERATED PORE PRESSURE
FOR CPTU SOUNDING P-17, NEW YORK CITY

CONE PENETRATION TEST



A AND B ARE SANDY UNITS

STRATIGRAPHICS

FIGURE 7 - GENERATED PORE PRESSURE VERSUS CONE RESISTANCE FOR CPTU SOUNDING D-1, WINTHROP HARBOR, ILLINOIS

Change of Hydraulic Head with Depth

By pausing in the penetration process and allowing generated pore pressures to dissipate to equilibrium conditions, the potentiometric surface (as would be measured in a fixed piezometer) is obtained at any particular depth. Therefore, vertical gradients and hydraulic head relationships can be investigated in one CPTU sounding without the significant time and cost expenditure associated with a cluster of fixed piezometers.

At the landfill in northeastern Illinois, it was important to determine if the uppermost aquifer discharged to a nearby swamp. CPTU Sounding D-1 indicated that the hydraulic head in a discontinuous sand unit (indicated by "A" in Figure 7) above the aquifer, was lower than the surface elevation of the overlying swamp. Also, the head in this unit was 0.5 feet greater than the hydraulic head in the aquifer (indicated by "B" in Figure 7) itself. These data indicate a downward gradient, showing that the swampy area of Sounding D-1 was a recharge area for the aquifer, rather than being a discharge area from the aquifer to the swamp.

The determination of lateral flow is also important in that it identifies the sand unit as a continuous aquifer with lateral extent. Areas of lateral flow are indicated by constant hydraulic heads with depth. In CPTU Sounding D-4 (Figure 8) at the landfill in northeastern Illinois, the hydraulic head measured at different depths in the uppermost aquifer was approximately the same, indicating a region of predominantly lateral flow.

The CPTU piezometric sounding is useful not only in locating the aquifers but in determining the head in confined aquifers before installing wells. The pore pressure in Sounding D-6 indicated that the head in the aquifer was just above the top of the unit (Figure 9). This low head was confirmed by the drilled monitoring well which was installed based on the CPTU-defined stratigraphy.

Direction of Groundwater Flow and Gradient

The United States Environmental Protection Agency generally requires a minimum of one up-gradient well and three down-gradient wells to monitor a landfill. In many cases the direction of groundwater flow is not known and must be determined before the monitoring well system can be designed and developed. Usually, several drilled test wells or piezometers are installed and surveyed to obtain this information. Groundwater levels are measured after the wells/piezometers have stabilized. This process can take days or weeks, depending upon site conditions. If the terrain is suitable and the water table is within the depth capability of the equipment, use of CPTU can result in a more detailed characterization and at less cost than a drilled investigation. A map of the water table or potentiometric surface can be rapidly made from the CPTU data utilizing surveyed ground surface elevations of the sounding locations.

CONE PENETRATION TEST

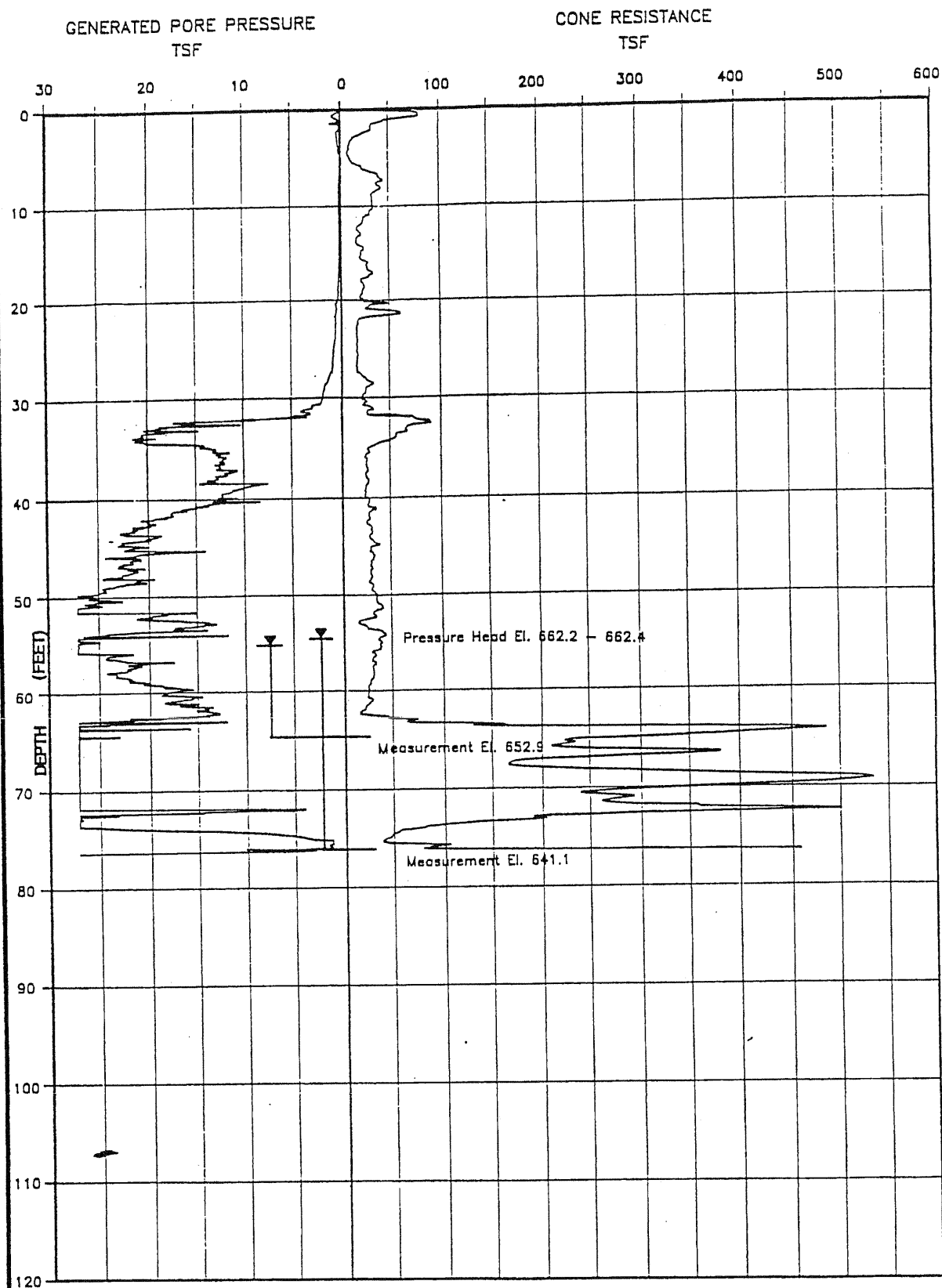
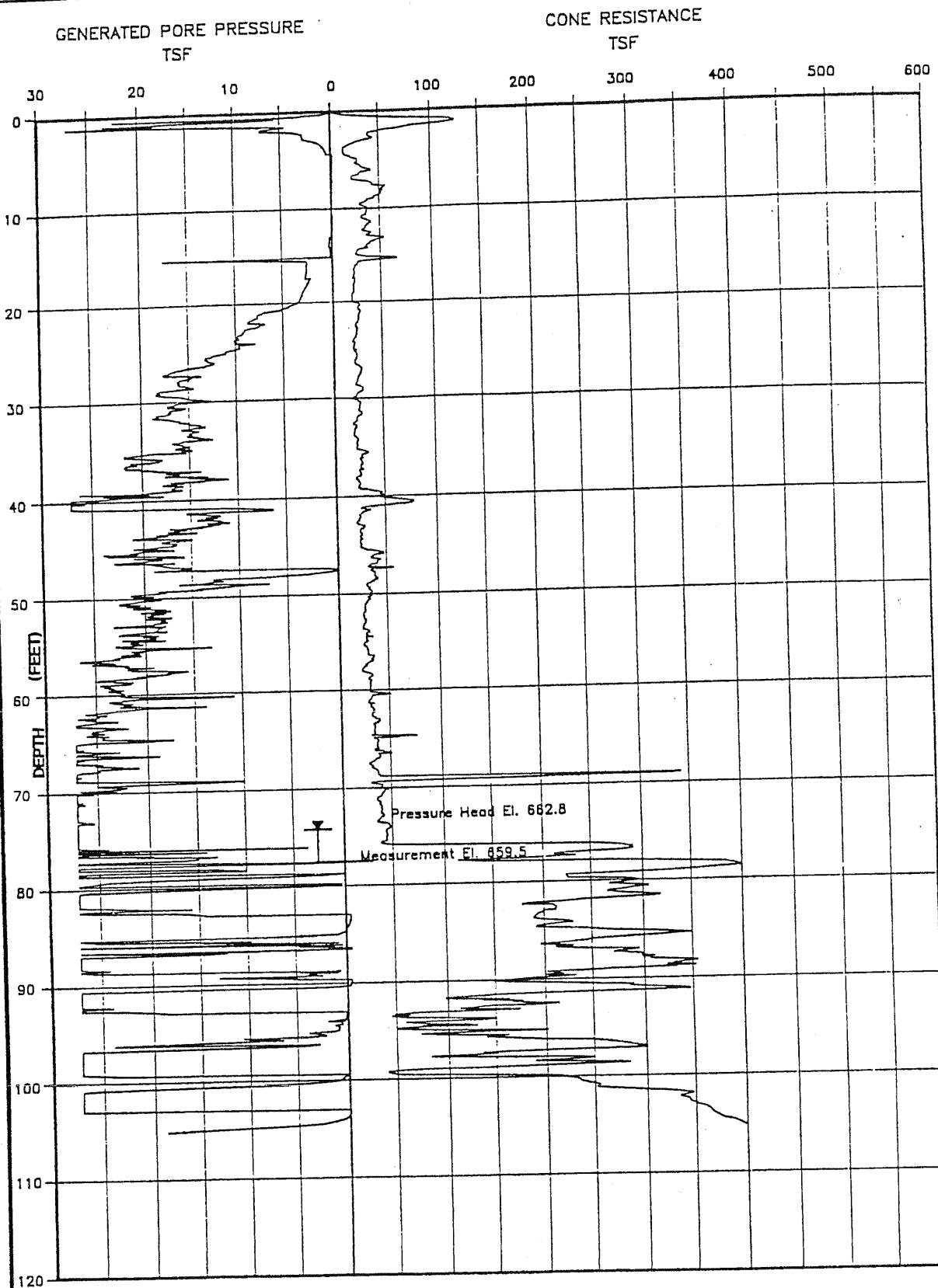


FIGURE 8 - GENERATED PORE PRESSURE VERSUS CONE RESISTANCE FOR CPTU SOUNDING D-4,
WINTHROP HARBOR, ILLINOIS

CONE PENETRATION TEST



STRATIGRAPHICS

FIGURE 9 — GENERATED PORE PRESSURE VERSUS CONE RESISTANCE FOR CPTU SOUNDING D-6, WINTHROP HARBOR, ILLINOIS

The gradient of the water table or potentiometric surface is related to the transmissivity. Other factors being equal, areas of flatter gradient are generally more transmissive (Table 1). The areas of flatter gradient are evident from the water table or potentiometric map. The gradient can be calculated by dividing the head loss by the distance across which the head is lost. Along with the other hydrogeological information the gradients derived from the CPTU data may be used in groundwater exploration before test drilling, as there is a direct relationship between transmissivity and water supply potential (well yield).

Aquifer Coefficients

Permeability and Transmissivity Estimates. An estimate of the permeability can be made from the CPTU classification of the soil encountered. For example, in CPTU Sounding D-6 (Figure 4) at the northeastern Illinois landfill, a silty sand aquifer was identified below 76 feet. According to Tables 2 and 3, the permeability of silty sands is in the range of 1.0×10^{-3} to 1.0×10^{-5} cm/sec. A slug test conducted subsequently in a drilled monitoring well constructed at the D-6 location, indicated a permeability of 1.8×10^{-4} cm/sec, which was within the estimated range.

By using permeability values estimated from CPTU soil classifications, and multiplying by the thickness of the unit as determined by the CPTU sounding, an approximate value of transmissivity can be obtained before drilling and pump testing. This estimate of transmissivity can be an important factor in deciding whether or not to drill a test well at the sounding location during a groundwater exploration investigation.

Pore Pressure Dissipation Permeability Values. By pausing during the penetration process, and allowing the pore pressures generated by penetrometer insertion to dissipate with time, a horizontal permeability value at the depth of the penetrometer can be obtained based on the measured time rate of dissipation. Computed permeabilities from CPTU pore pressure dissipations are representative of the disturbed soil immediately in contact with the CPTU piezometric filter. This zone has a vertical extent of only 1/4 to 1 inch (0.6 - 2.5 cm). An assumption of soil compressibility is also required to compute permeability from CPTU data.

Examples of measured dissipation curves are provided in Figure 10. The evaluation of horizontal permeability is performed using normalized dissipation plots, as shown in Figure 11, and procedures detailed in Baligh and Levandoux (1980) or Robertson et al (1986). The normalized dissipation data (Figure 11) from 20.5 and 50.0 feet were obtained in the silty clay tills in CPTU Sounding D-1 at the landfill in northeastern Illinois. These data indicate horizontal permeabilities of about 1.0×10^{-6} cm/sec in the silty clay till. This value is in good agreement with slug test results in monitoring wells finished in the same stratum.

TABLE 1
AQUIFER HORIZONTAL GRADIENTS

.0005 - .002	good aquifers
about .01	poor aquifers
>.05	aquitards

Source: Mandel and Shiftan (1981, p. 172).

TABLE 2
ESTIMATED HYDRAULIC CONDUCTIVITY OF
TYPICAL GEOLOGIC MATERIALS IN ILLINOIS

Geologic material	cm/sec	gpd/ft ²	Comments
Clean sand and gravel	$> 1 \times 10^{-3}$	> 20	May be highly permeable
Fine sand and silty sand	1×10^{-5} to 1×10^{-3}	0.2 to 20	—
Silt (loess, colluvium, etc.)	1×10^{-6} to 1×10^{-4}	1×10^{-1} to 2	—
Gravelly till, less than 10% clay	1×10^{-7} to 1×10^{-5}	2×10^{-3} to 2×10^{-1}	Often contains gravel/sand lenses or zones
Till, less than 25% clay	1×10^{-8} to 1×10^{-6}	2×10^{-4} to 2×10^{-2}	Often contains gravel/sand lenses or zones
Clayey tills, greater than 25% clay	1×10^{-9} to 1×10^{-7}	2×10^{-5} to 2×10^{-3}	Often contains gravel/sand lenses or zones
Sandstone	$> 1 \times 10^{-4}$	> 2	—
Cemented fine sandstone	1×10^{-7} to 1×10^{-4}	2×10^{-3} to 2	Frequently fractured
Fractured rock	$> 1 \times 10^{-4}$	> 2	May have extremely high hydraulic conductivity
Shale	1×10^{-11} to 1×10^{-7}	2×10^{-7} to 2×10^{-3}	Often fractured
Dense limestone/dolomite (unfractured)	1×10^{-11} to 1×10^{-8}	2×10^{-7} to 2×10^{-4}	—

Source: Berg and Others (1984)

TABLE 3
RELATIVE VALUES OF PERMEABILITY

<u>Relative Permeability</u>	<u>Values of k (cm/sec)</u>	<u>Typical Soil</u>
Very permeable	Over 1×10^{-1}	Coarse gravel
Medium permeability	1×10^{-1} to 1×10^{-3}	Sand, fine sand
Low permeability	1×10^{-3} to 1×10^{-5}	Silty sand, dirty sand
Very Low permeability	1×10^{-5} to 1×10^{-7}	Clay
Impervious	Less than 1×10^{-7}	Clay

Source: Sowers and Sowers (1970, p.93)

TABLE 4
COMPARISON OF TRANSMISSIVITY, SPECIFIC CAPACITY,
AND WELL POTENTIAL

<u>TRANSMISSIVITY</u>									
<u>FT³/FT/DAY (ft²/day)</u>									
10^8	10^7	10^6	10^5	10^4	10^3	10^2	10^1	1	10^{-1}
<u>FT³/FT/MIN (ft²/min)</u>									
10^4	10^3	10^2	10^1	1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}
<u>GAL/FT/DAY (gal/ft/day)</u>									
10^8	10^7	10^6	10^5	10^4	10^3	10^2	10^1	1	10^{-1}
<u>METERS³/METER/DAY (m²/day)</u>									
10^6	10^5	10^4	10^3	10^2	10^1	1	10^{-1}	10^{-2}	10^{-3}
<u>SPECIFIC CAPACITY (gal/min/ft)</u>									
10^5	10^4	10^3	10^2	10^1	1	10^{-1}	10^{-2}	10^{-3}	10^{-4}
<u>WELL POTENTIAL</u>									
<u>Irrigation</u>					<u>Domestic</u>				
UNLIKELY	VERY GOOD	GOOD	FAIR	POOR	GOOD	FAIR	POOR	INFEASIBLE	

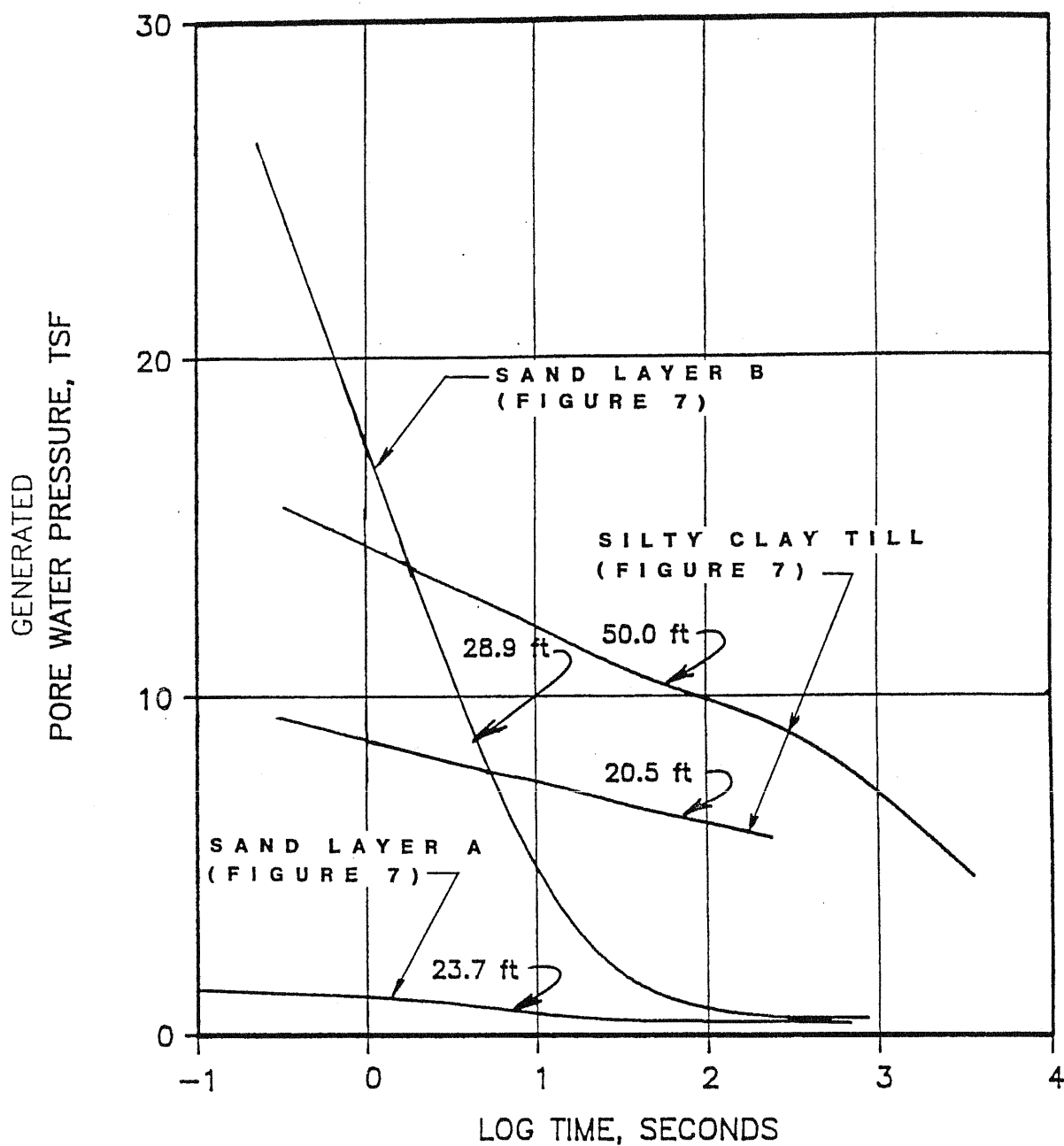
NOTES: Transmissivity (T)=KM where

K=Permeability

M=Saturated thickness of the aquifer

Specific capacity values based on pumping period of approximately 8-hours but are otherwise generalized.

Source: U.S. Bureau of Reclamation (1977, Figure 2-4)



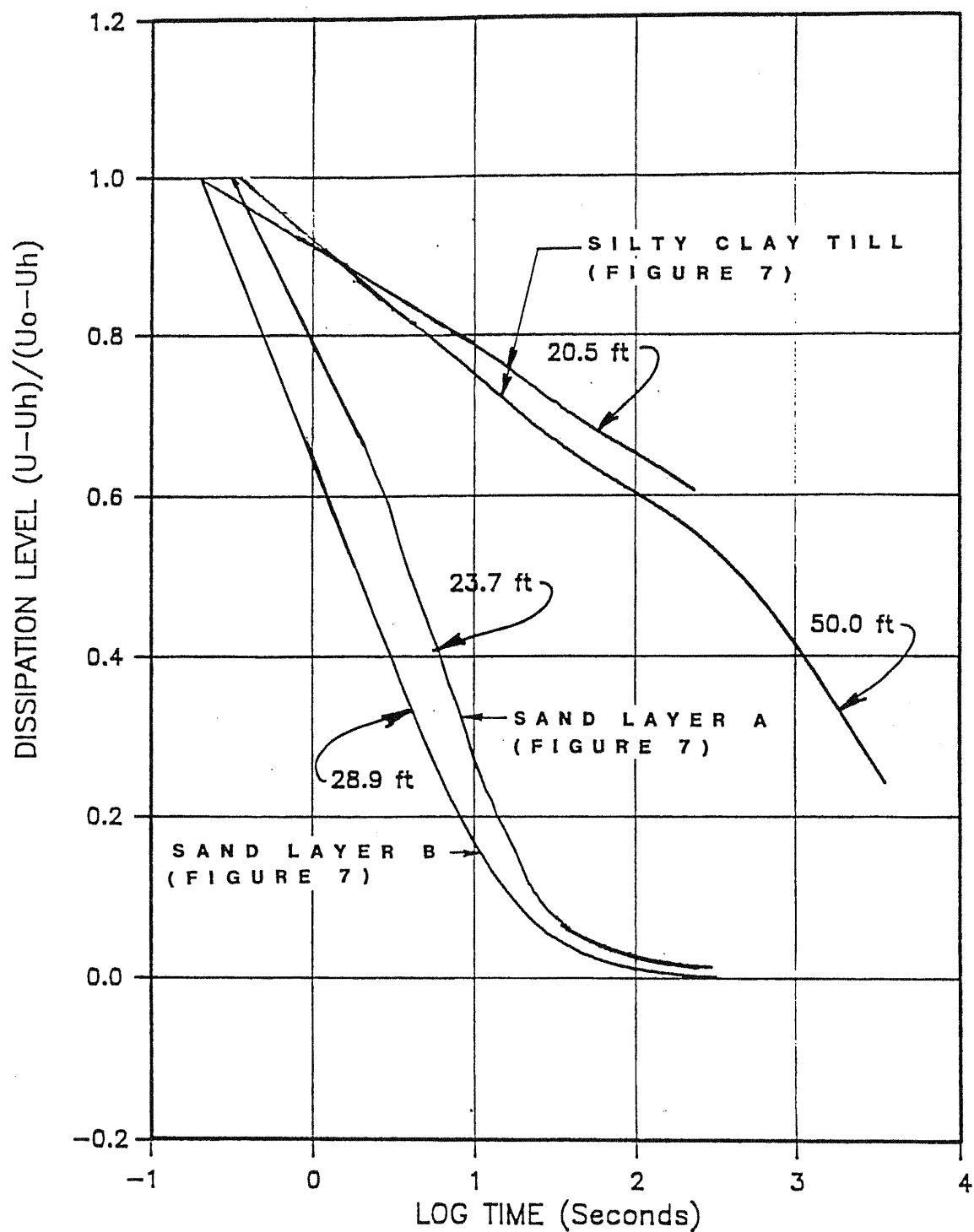
CPT SOUNDING D-1

STRATIGRAPHICS

CPT DISSIPATION DATA

LAKE COUNTY, IL

FIGURE 10 — GENERATED PORE WATER PRESSURE, CPTU SOUNDING D-1, WINTHROP HARBOR, ILLINOIS



CPT SOUNDING D-1

STRATIGRAPHICS

CPT DISSIPATION DATA

LAKE COUNTY, IL

FIGURE 11 - PORE WATER PRESSURE DISSIPATION LEVEL, CPTU SOUNDING D-1, WINTHROP HARBOR, ILLINOIS

The other dissipation data in Figure 11 were obtained in a discontinuous silty sand layer in CPTU Sounding D-1, at 23.8 feet, and in the silty sand uppermost aquifer at 28.9 feet. These horizontal permeabilities are indicated by the CPTU dissipation data to range from about 1.5×10^{-5} to 7.0×10^{-5} cm/sec. A slug test in the uppermost aquifer in a monitoring well drilled at location D-1 indicated a permeability of about 2.0×10^{-5} cm/sec, in the lower range of values determined from CPTU. The monitoring well screen, at a depth of 27 to 35 feet, was set opposite interbedded silty sand and silty clay layers.

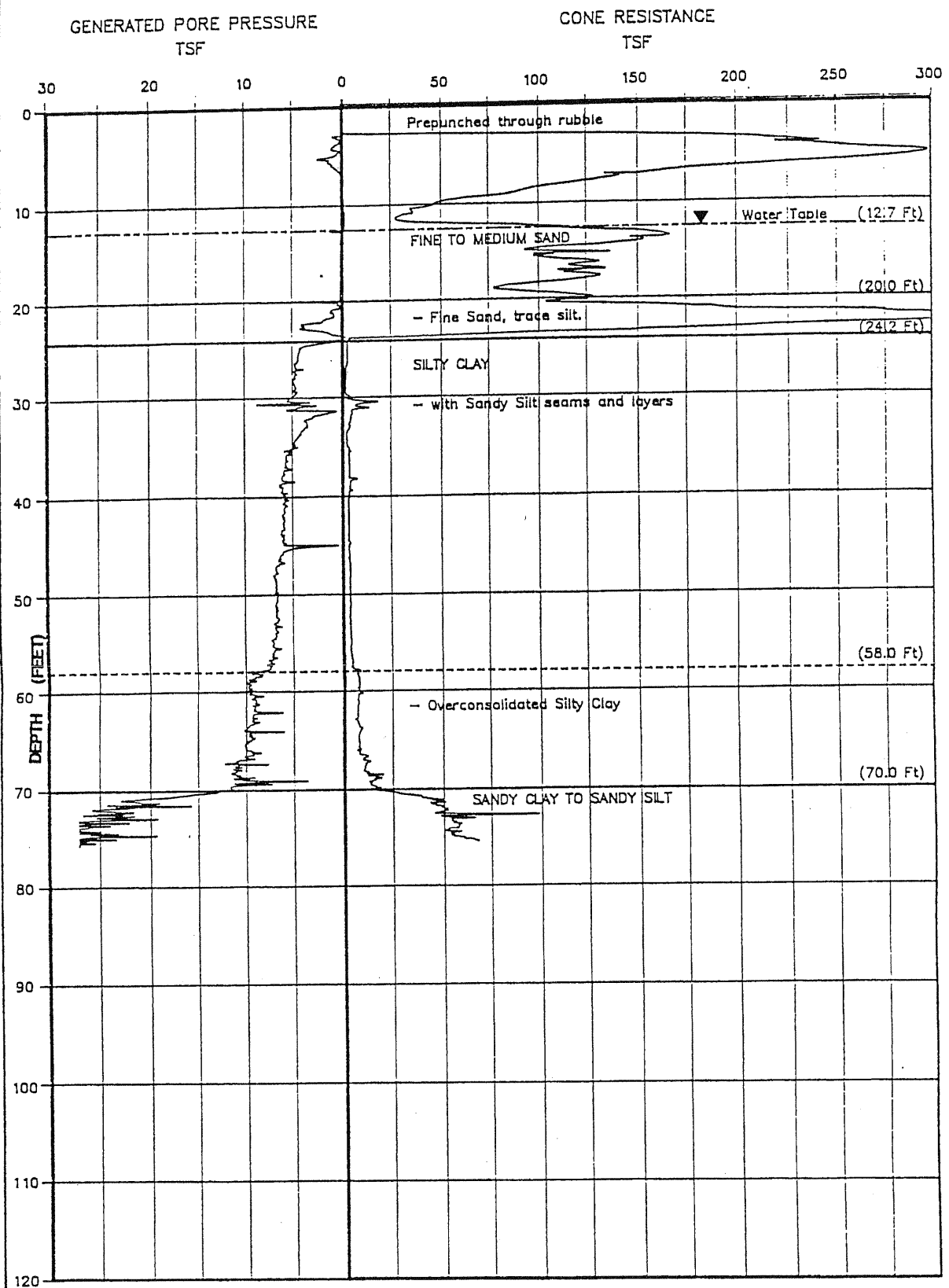
Example of Estimation of Potential Well Yield From CPTU Sounding Data

A potential well yield of at least 2.0 gpm (0.13 liters/sec) was estimated from CPTU data in a thin water table sand in Evanston, Illinois. Interpretation of the CPTU Sounding NW-1 (Figure 12) indicated a saturated, clean, fine to medium sand from 12.7 to 20 feet below the surface. Based on Tables 2 and 3, the permeability of this zone was estimated as about 5.0×10^{-3} cm/sec. Multiplying the saturated thickness by the permeability gives a transmissivity (T) of 10 square meters/day or 800 gpd/ft. According to Table 4, this T is in the range of good aquifers for domestic supplies, but not for irrigation supplies. The specific capacity is estimated from Figure 13. Using a water table coefficient of storage of 0.2 and a T of 800 gpd/ft, the specific capacity was estimated as 0.8 gpm/ft of drawdown.

The lower third of a water table aquifer is screened in typical installations. A better design in this thin aquifer would be installing a 5-foot screen with a pump set one foot above the bottom. Allowing one foot for the pump and 3 feet to cover the pump, the remaining available drawdown is only 2.5 feet. Multiplying by the specific capacity of 0.8 gpm/ft indicates an estimated yield of 2.0 gpm.

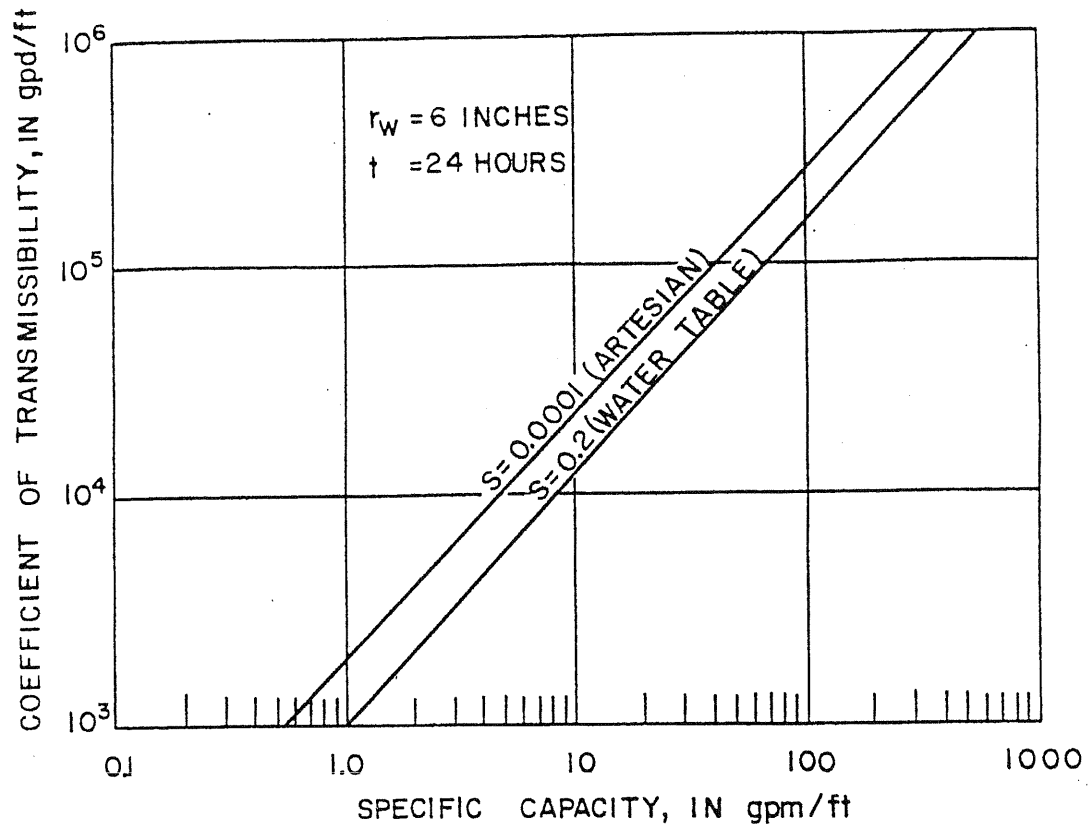
Examination of the CPTU Sounding NW-1 (Figure 12) indicates 4 feet of fine sand and silt below the aquifer which could also contribute to the well yield. Therefore, the estimate is considered conservative. By prior estimation of potential well yields using CPTU data, decisions regarding the locations of expensive test wells and pumping tests can be more intelligently made.

PIEZOMETRIC CONE PENETRATION TEST



STRATIGRAPHICS

FIGURE 12—GENERATED PORE PRESSURE VERSUS CONE RESISTANCE FOR CPTU SOUNDING NW-1, EVANSTON, ILLINOIS



SOURCE: WALTON (1970, FIG. 5.8)

FIGURE 13 - GRAPH OF SPECIFIC CAPACITY VERSUS TRANSMISSIVITY AFTER 24 HOURS OF PUMPING

Installation of Piezometers in CPTU Sounding Holes

Small diameter, standpipe piezometers may be installed in CPTU sounding holes, in many places, at a significant savings of time and money compared to drilling methods. Four installation methods are available for placing standpipe piezometers using CPTU equipment. These four methods are: 1) pushed 1 1/4-inch (nominal) steel well point, 2) PVC riser in temporarily cased hole, 3) PVC riser in open hole, and 4) pushed 2-inch (nominal) steel well point.

Of the four methods, the 1 1/4-inch (nominal) steel well point is most preferred. The advantages of this installation technique are: 1) depth of installation nearly equal to the depth of a CPTU sounding; 2) rapid installation with minimal personnel contact with possibly contaminated soils; 3) exceptional sealing of risers to formation, with excellent isolation of well screen; and 4) low labor cost due to rapid installation with moderate material cost. One disadvantage of this technique is that no sand or gravel pack can be used as a well screen filter other than that which may naturally occur at the screened interval. Another is the diameter, which is too small for a submersible pump.

The second CPTU standpipe piezometer installation technique consists of the following steps: a small diameter hole is punched to the desired depth, either during CPTU, or by using an uninstrumented conical tip. A 2-inch (nominal) steel casing is then pushed down the pre-punched hole to the required depth. The end of this casing is closed with a slip-on cap, which remains in the soil after casing withdrawal. Three-quarter inch (nominal) PVC well screen and risers are lowered to the bottom of the casing. Sand pack is poured down the annulus, tamped with a long thin steel rod around the well screen, as the casing is slowly withdrawn, leaving the riser down-hole. A bentonite seal is placed above the sand pack to isolate the well point. The advantages of this technique are: 1) low cost for PVC well materials; 2) moderate labor time during installation; and 3) hole kept open by temporary casing. A disadvantage of this technique is that the completion depth is limited due to the use of the relatively large diameter 2-inch casing. The maximum depth of installation depends on site stratigraphy, but is certainly much less than the depth that can be achieved with the CPTU sounding itself. Also the small diameter riser (0.83-inch ID) makes the well difficult to develop if the water level is deeper than the lift of a suction pump. It must then be developed and sampled with a bailer.

The third technique, PVC riser in open hole, is the least expensive, both in installation time and materials, but is potentially also the least reliable. A 3/4-inch or 1-inch (nominal) PVC well screen and riser is lowered into the open hole left by the penetrometer. In squeezing soils, the hole may be enlarged using a 2-inch uninstrumented cone tip. After the PVC is lowered, sand pack is poured down the annulus and tamped around the screen, followed by a bentonite seal.

Advantages to this technique are: 1) low labor cost; 2) very low material cost; and 3) rapid installation. The disadvantage of this technique is that installation in an open hole is less reliable than in a cased hole. Caving sands or squeezing clays can affect both sand pack and grout seal. Caving sands and squeezing clays can also preclude deep installations. Again the small diameter of the riser pipe is a disadvantage.

The fourth technique of setting fixed piezometers with CPTU equipment involves pushing a 2-inch (nominal) steel pipe and well screen to the required depth. An enlarged hole is pre-punched with a 2-inch uninstrumented cone tip to facilitate well riser insertion. Advantages to this method are that the inner diameter of the riser is greater than 2 inches, and moderate labor and material costs. The diameter allows use of a submersible pump for development and water sampling. The disadvantage to this technique is limited depth capacity due to the relatively large diameter of the riser.

The ability to install fixed piezometers or small diameter wells in CPTU sounding holes gives the CPTU method a powerful versatility. In many environmental and geotechnical projects, shallow piezometers can be located at the proper depth and installed with the CPTU rig, without bringing in a drilling rig. In groundwater supply projects for domestic and rural village water supplies, sand aquifers can be located, estimates of potential well yield can be made, and steel well points installed quickly by using CPTU equipment alone, without drilling rigs.

Testing and Sampling. Standpipe piezometer installations with 2-inch (5 cm) risers can be developed and sampled and pump tested with a submersible pump. Standpipe piezometer installations with risers at least one inch (2.54 cm) in inside diameter are readily developed, tested, and sampled with a bailer. After development, a bailer or slug test may be conducted to determine the permeability of the material opposite the screen. Water samples may be bailed from the well for chemical analysis. The 0.83-inch ID (3/4-inch nominal) PVC riser can also be bailed with a specially made slim bailer. Small diameter piezometers may be pumped if the water level is shallow enough, and the intake pipe of a suction pump can fit inside the riser. Water levels, of course, can readily be obtained in all of these standpipe piezometers.

COST COMPARISON BETWEEN CPTU AND DRILLING AND SAMPLING

CPTU can be an efficient tool to obtain critical geotechnical and hydrogeological information. The key element is that the CPTU sounding provides a continuous log of the subsurface soil conditions and properties. To obtain a continuous log of subsurface conditions using traditional drilling techniques is costly and time consuming. For example, 100 feet of data using CPTU or drilling and sampling may cost approximately:

- A) \$1,000-\$1,400 for CPTU, with interpretation, pore pressure dissipation, and hole grouting
- B) \$8,000-\$10,200 for continuously sampled boring/analysis (including laboratory testing)
- C) \$1,850-\$3,900 for conventional boring/analysis (non-continuous sampling/testing at 5-foot intervals)

The drilling/boring costs do not include a field technician which may add \$500 to \$750 to the cost of Alternatives B and C. Thus, CPTU may result in a cost savings of about 85 percent as compared to continuous boring/sampling/analysis and about 35 to 65 percent when compared to the cost of conventional boring/sampling/analysis.

Use of CPTU saved an estimated \$4,000 (U.S.) in the monitoring well installation program for the landfill project in northeastern Illinois. By having the continuous CPTU sounding logs defining site stratigraphy, and knowing exactly at what depth the aquifer would be encountered, it was possible to install the monitoring wells using hollow stem augers, without expensive continuous sampling. Additionally, fewer geotechnical laboratory tests on the samples were required, and less geological supervision and logging time was necessary to complete the well installations.

CPTU standpipe piezometer installations are also less expensive than drilled piezometers. Both CPTU and drilled borehole techniques were used at another site in northern Illinois to provide data for site characterization and for shallow piezometer installations. The total cost for the drilled portion of the investigation was about \$6,900; the cost for about the same amount of work based on CPTU methods was only \$3,300, or less than half of the drilled cost. The drilling cost did not include the cost of the geologist who supervised the drilling and logged the samples. No geological supervision was required with the CPTU method, resulting in additional savings.

CONCLUSIONS

Piezometric Cone Penetration Testing (CPTU) provides a cost effective, accurate, and rapid means to determine hydrogeologic properties at suitable sites, including stratigraphy, saturation, hydraulic head, lateral gradient, vertical gradient, position of the water table, position of the potentiometric surface, slope of the water table, direction of groundwater movement, and permeability and transmissivity. This information is readily defined in a short period of time using computerized data acquisition techniques. Data are objective, and analysis is straight forward as CPTU test results directly reflect soil characteristics of grain size, void ratio and permeability. Costs of CPTU can be as much as 85 percent less than costs associated with a drilled borehole with continuous sampling and laboratory testing.

Small diameter, standpipe piezometers can also be installed using CPTU equipment after completion of a sounding. At contaminated sites, lessened personnel exposure and greatly decreased generation of cuttings and drilling fluids characterize CPTU exploration techniques.

REFERENCES

Baligh, M.M., and Levadoux, J.N., 1980, "Pore Pressure After Cone Penetration," Department of Civil Engineering, MIT, Cambridge, Massachusetts.

Berg, R.C., Kempton, J.P., and Cartwright, Keros, 1984, "Potential for Contamination of Shallow Aquifers in Illinois," Illinois State Geological Survey, Circular 532.

Douglas, B.J., and Olsen, R.S., 1981, "Soil Classification Using the Electric Cone Penetrometer," Cone Penetrometer Testing and Experience, ASCE Special Technical Publication, St. Louis, Missouri.

Mandel, S. and Shiftan, Z.L., 1981, Groundwater Resources - Investigation and Development, Academic Press, Inc., Orlando, Florida, 269p.

Olsen, R.S. and Farr, J.V., 1986, "Site Characterization Using Cone Penetrometer Test," Use of In Situ Tests in Geotechnical Engineering, ASCE Geotechnical Special Publication No. 6, Blacksburg, Virginia.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data," Use of In Situ Tests in Geotechnical Engineering, ASCE Geotechnical Special Publication No. 6, Blacksburg, Virginia.

Sowers, G.B. and Sowers G.F., 1970, Introductory Soil Mechanics and Foundations, 3rd Edition, The MacMillan Company, New York, 556p.

Strutynsky, A.I., Douglas, B.J., Mahar, L.J., Edmonds, G.F. and Hencey, C., 1985, "Arctic Penetration Test Systems," Civil Engineering in the Arctic Offshore, ASCE Conference Proceedings, Alaska.

U.S. Bureau of Reclamation, 1977, Ground Water Manual, Washington, D.C., 480p.

Todd, D.K., 1980, Groundwater Hydrology, 2nd Edition, John Wiley and Sons, New York, 535p.

Walton, W.C., 1970, Groundwater Resource Evaluation, McGraw Hill, New York, 664 p.